STRUCTURAL ROBUSTNESS OF STEEL FRAMED BUILDINGS

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STRUCTURAL ROBUSTNESS OF STEEL FRAMED BUILDINGS

STRUCTURAL ROBUSTNESS OF STEEL FRAMED BUILDINGS

In accordance with Eurocodes and UK National Annexes

A G J Way MEng CEng MICE

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FOREWORD

In the UK, the design of hot-rolled steel-framed buildings for avoidance of disproportionate collapse and robustness has, since 1990, generally been in accordance with the British Standard BS 5950-1. However, that Standard was withdrawn in March 2010 and replaced by the corresponding Parts of the Eurocodes. The main Eurocode Part that will need to be consulted for design to resist accidental actions and strategies to achieve structural robustness is BS EN 1991-1-7 and its National Annex.

This guide was prepared to describe the design of hot-rolled steel buildings for structural robustness in accordance with the Eurocodes. Its scope is similar to that of SCI publication P341, *Guidance on meeting the Robustness Requirements in Approved Document A*, which focussed on the UK Regulations and offered guidance in relation to BS 5950-1.

This publication does not specifically deal with accidental actions caused by external explosions, warfare and terrorist activities, or the residual stability of buildings or other civil engineering works damaged by seismic action or fire.

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CONTENTS

SUMMARY

This publication provides design guidance for hot-rolled steel-framed buildings on the Eurocode strategies for structural robustness and designing for the avoidance of disproportionate collapse as required by the UK Building Regulations.

Design guidance in accordance with the Eurocodes is presented for the four building classes in the Eurocodes and the UK Regulations. Guidance on recommended good practice is presented where the Eurocodes do not give requirements or where they are not specific and are open to interpretation. The scope of this publication is limited to application in the UK and reference is made to the UK National Annexes as appropriate.

In addition to the design guidance, six worked examples are included to demonstrate the application of robustness strategies to different classes of building. Detailed guidance on the building classification process is also presented.


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INTRODUCTION

1.1 Scope

This publication provides design guidance on the Eurocode and UK Building Regulation requirements for providing ‘robustness’ and the avoidance of disproportionate collapse in hot-rolled steel framed buildings. Guidance is provided on how the requirements relate to common structural solutions. Recommended practice is presented where the Eurocodes do not give requirements or where the requirements are not specific or are open to interpretation.

The majority of the robustness related clauses are given in BS EN 1991 [1] and in particular Part 1-7. However, reference is also made to BS EN 1993 [2] and BS EN 1990 [3], as appropriate.

The scope of this publication is limited to application in the UK. Reference is made to the UK National Annexes as appropriate.

This publication does not address the other legal obligations that are related to robustness. However, designers should be aware of such obligations and the implications for design. The relevant references include; Building Act [4], Health and Safety at Work Act [5], Construction (Design and Management Regulations) [6] and Workplace (Health, Safety and Welfare) Regulations [7]. Guidance on these is provided in Reference 24.

1.2 Eurocodes

There are ten Eurocodes, which together provide a comprehensive set of Standards covering all aspects of structural design using the normal construction materials. The ten Eurocodes are:

- BS EN 1990 - Eurocode: Basis of structural design;
- BS EN 1991 - Eurocode 1: Actions on structures;
- BS EN 1992 - Eurocode 2: Design of concrete structures;
- BS EN 1993 - Eurocode 3: Design of steel structures;
- BS EN 1994 - Eurocode 4: Design of composite steel and concrete structures;
- BS EN 1995 - Eurocode 5: Design of timber structures;
- BS EN 1996 - Eurocode 6: Design of masonry structures;
- BS EN 1997 - Eurocode 7: Geotechnical design;
- BS EN 1998 - Eurocode 8: Design of structures for earthquake resistance;
- BS EN 1999 - Eurocode 9: Design of Aluminium Structures.

Each Eurocode is comprised of a number of Parts, which are published as separate documents. For a general introduction to the Eurocodes in relation to the design of steel buildings, see SCI publication P361 [8].

National Annexes

Within the full text of a Eurocode, national choice is allowed in the setting of some factors and in the choice of some design methods. The choices are generally referred to as Nationally Determined Parameters (NDP) and these are published in a National Annex. Each part of the Eurocodes has a separate National Annex.

The guidance given in a National Annex applies to structures that are to be constructed within that country. National Annexes are likely to differ between countries within Europe. The National Annex for the country where the structure is to be constructed should always be consulted in the design of a structure.

Within this publication, the values and choices recommended in the UK National Annexes are used.

1.3 Robustness

The term robustness is often used generically to infer properties such as sturdiness, strength, solidity and durability. However, in Eurocodes, robustness has a precise definition and it is in the context of the Eurocode definition that the term robustness is used in this publication. Robustness is defined in BS EN 1991-1-7 *Actions on structures. General actions. Accidental actions*, as follows:

> *Robustness is the ability of a structure to withstand events like fire, explosions, impact or the consequences of human error, without being damaged to an extent disproportionate to the original cause.*

From this definition it can be concluded that a structure designed and constructed to have robustness will not suffer from disproportionate collapse. Design for avoidance of disproportionate collapse is a requirement of Building Regulations in the UK (see Section 2).

Accidental design situations are defined in BS EN 1990, reproduced below; from this it can be seen that the events referred to in the BS EN 1991-1-7 definition of robustness are accidental design situations.

> *Accidental design situations are design situations involving exceptional conditions of the structure or its exposure, including fire, explosion, impact or local failure.*

In essence, the objective is to ensure that buildings do not suffer disproportionate collapse under accidental loading. Largely, this is assured in steel framed buildings by designing connections appropriately.

The terms disproportionate collapse and progressive collapse are often used interchangeably but it is possible to make a distinction. Progressive collapse is the spread of structural collapse from the initial failure of one or a few localised structural elements. If progressive collapse occurs it does not necessarily result in disproportionate collapse. However, the Ronan Point collapse illustrates a case where progressive collapse did result in disproportionate collapse (see Figure 1.1). The Ronan Point collapse was the motivation for introducing disproportionate collapse regulations in the UK and is well documented; for further information see References 9 and 10.

Figure 1.1 Ronan Point 1968 - Partial collapse of a concrete structure due to a gas explosion

BUILDING REGULATIONS

2.1 The 'Requirement'

In the UK, there are three different sets of Building Regulations, one for each of the following jurisdictions:

1. England and Wales;
2. Scotland;
3. Northern Ireland.

Although the wording varies slightly, the 'Requirement' concerning disproportionate collapse is essentially the same in all three jurisdictions.

Requirement A3 from Part A of the England and Wales Building Regulations [11] is given below.

> *The building shall be constructed so that in the event of an accident the building will not suffer collapse to an extent disproportionate to the cause.*

The phrase "*in the event of an accident*" from the requirement given above, ties up neatly with the phrase "*consequences of human error*" from the definition of robustness given in BS EN 1991-1-7. Both of these phrases help to emphasize that we are not dealing with deliberate acts intended to cause damage or collapse. However, all buildings should be robust and the engineer has a responsibility to consider all loads including those due to malicious action. In most cases, malicious acts have a very low risk of occurrence and can reasonably be discounted. However, there might be occasions where their consideration is warranted. This matter will be part of the overall consideration of hazard and risk and should be discussed with the client.

At the time of writing, the avoidance of disproportionate collapse requirement under the Building Regulations is applicable as follows:

1. In England and Wales - Applicable to all buildings.
2. In Scotland - Applicable to all buildings.
3. In Northern Ireland - Applicable to all buildings with five or more storeys.

2.2 Guidance documents

In each of the three jurisdictions listed above, official guidance documents are published to explain how compliance with the regulatory requirements may be achieved.

In England and Wales, the guidance documents are termed Approved Documents and there is one for each part of the Regulations. Approved Document A [12] includes guidance on how the key robustness requirement A3 should be applied to different types and sizes of building.

In Scotland, guidance on satisfying the regulations is given in The Scottish Building Standards Agency (SBSA) Technical Handbooks [13].

In Northern Ireland, guidance on satisfying the regulations is given in The Building Regulations (Northern Ireland), Technical Booklet D [14].

In due course, these official guidance documents will be updated to reference the Eurocodes, where guidance on designing steel framed structures to avoid disproportionate collapse is given (see Section 3).

2.3 Approved Document A

2.3.1 Building classes

Approved Document A (2004 Edition) [12] sets out different required levels of robustness for different types and sizes of buildings. There are four classes of building; Class 1, Class 2A, Class 2B and Class 3. The building classification presented in Approved Document A is the same as that presented in the SBSA Technical Handbooks [13] and there is only a small difference from that presented in BS EN1991-1-7. The classification of buildings is described in detail in Section 4.

The robustness requirements specified in Approved Document A for each class of building are summarised below. Detailed guidance and explanations of the structural provisions that should be provided for each of the building classes are given in Sections 5, 6, 7 and 8.

For Class 1 buildings

Provided that the building has been designed and constructed in accordance with the rules given in Approved Document A for normal use, no additional measures are likely to be necessary.

For Class 2A buildings

Effective horizontal ties should be provided for framed construction.

For Class 2B buildings:

There are three methods by which the robustness requirements may be satisfied for Class 2B buildings.

1. Provide effective horizontal ties, together with effective vertical ties in all supporting columns.
2. Check that upon the notional removal of a supporting column or a beam supporting one or more columns (one at a time in each storey of the building)

that the building remains stable and that the area of floor at any storey at risk of collapse does not exceed 15% of the floor area of that storey or 70 m^2, whichever is smaller, and does not extend further than the immediate adjacent storeys. (Note: The limit is 100 m^2 in BS EN 1991-1-7 and it is expected that at its next revision the limit in Approved Document A will be revised to agree with BS EN 1991-1-7.)

3. Where the notional removal of such columns (or beams supporting one or more columns) would result in damage in excess of the above limit, then such elements should be designed as key elements.

Note: At the time of writing, there is ongoing debate about whether Approved Document A should be amended such that the provision of horizontal ties is recommended in all cases of Class 2B buildings irrespective of which method is adopted to satisfy the robustness requirements. Reference 24 makes the point that the notional removal and key element methods are principally concerned with vertical structure or elements supporting vertical structure. Therefore, when applying the notional removal or key element methods, the designer must still ensure that the structure is robust in orthogonal horizontal directions, which is generally achieved by providing horizontal ties.

For Class 3 buildings

A systematic risk assessment of the building should be undertaken, taking into account all the normal hazards that can reasonably be foreseen, together with any abnormal hazards. Critical situations for design should be selected that reflect the conditions that can reasonably be foreseen as possible during the life of the building.

2.3.2 Eurocodes and Approved Document A

Approved Document A permits any method of design to be used, providing it satisfies the functional requirements of the Building Regulations.

The current 2004 version of Approved Document A refers to national design standards, i.e. BS 5950, for the design of steel framed buildings and does not refer to Eurocodes. Approved Document A is scheduled for revision in 2013, at which point it will include reference to the Eurocodes.

In January 2010, CLG (Department of Communities and Local Government) issued a circular letter that confirmed the suitability of the Eurocodes to meet the requirements of the Building Regulations.

The following sections of this publication explain how steel framed buildings can be designed for structural robustness in accordance with the Eurocodes. Where it is deemed appropriate, additional structural provisions have been recommended.

EUROCODE ROBUSTNESS REQUIREMENTS

3.1 BS EN 1990

BS EN 1990 can be considered as the ‘core’ document of the structural Eurocode system, as it establishes the principles and requirements for the safety, serviceability and durability of structures. It also describes the basis for structural design and verification.

The main sections of BS EN 1990 include:

- Requirements.
- Principles of limit state design.
- Basic variables.
- Structural analysis and design assisted by testing.
- Verification by the partial safety factor method.

3.1.1 Basic requirements

BS EN 1990, 2.1 (1)P and (2)P sets out several basic requirements for the design of structures, including: the structure shall *“sustain all actions and influences likely to occur during execution and use”*; the structure shall *“be designed to have adequate structural resistance, serviceability and durability”*.

The principle given in BS EN 1990, 2.1 (4)P has particular relevance to structural robustness; it states that:

> *A structure shall be designed and executed in such a way that it will not be damaged by events such as: – explosion, – impact, and – the consequences of human errors, to an extent disproportionate to the original cause.*
>
> *Note 1 - The events to be taken into account are those agreed for an individual project with the client and the relevant authority.*
>
> *Note 2 - Further information is given in EN 1991-1-7.*

The wording in BS EN 1990, 2.1 (4)P is very similar to the definition of robustness given in BS EN 1991-1-7. In essence, the principle sets out the overriding requirement to provide a building that is designed and executed to have robustness.

The principle given in BS EN 1990, 2.1 (5)P also has particular relevance to structural robustness and states that:

> *Potential damage shall be avoided or limited by appropriate choice of one or more of the following:*
>
> - *avoiding, eliminating or reducing the hazards to which the structure can be subjected;*
> - *selecting a structural form which has low sensitivity to the hazards considered;*
> - *selecting a structural form and design that can survive adequately the accidental removal of an individual member or a limited part of the structure, or the occurrence of acceptable localised damage;*
> - *avoiding as far as possible structural systems that can collapse without warning;*
> - *tying the structural members together.*

These methods are structural philosophies rather than detailed design guidance. Detailed guidance is provided in BS EN 1991-1-7.

3.1.2 Design situations

The design situations that should be considered are given in BS EN 1990, 3.2. The consideration of accidental actions as a separate design situation is shown in BS EN 1990, 3.2 (2)P, which gives the following design situations:

- Persistent – during normal use of the structure.
- Transient – temporary conditions e.g. during execution or repair.
- Accidental – exceptional events e.g. exposure to fire, impact, explosion or consequences of localised failure.
- Seismic – conditions due to seismic events.

Information on specific design situations within each of these classes is given in the other Parts of the Eurocodes. Structural robustness is considered as part of the ‘Accidental’ design situation.

From BS EN 1990, 3.2(1)P and (3)P, it is clear that all relevant design situations should be selected and these should be sufficiently severe to include all reasonably foreseeable conditions during the intended use of the structure. Therefore, accidental design situations should be considered for all buildings designed in accordance with the Eurocodes.

3.1.3 Ultimate limit states

BS EN 1990 gives details of the ultimate limit states that must be verified and how design values of the effects of the actions should be combined. Detailed explanation of the limit states and load combinations for general application to steel building design is given in SCI publication P362 [15].

From BS EN 1990, 6.4.3.3, Eq. (6.11b) and Table NA.A1.3 of the UK National Annex, the combination of actions for accidental design situations can be expressed as:

$$\sum_{j\geq 1} G_{k,j} \text{ "+" } A_d \text{ "+" } \psi_{1,1} Q_{k,1} \text{ "+" } \sum_{i\geq 1} \psi_{2,i} Q_{k,i}$$

where:

"+"	implies "to be combined with"
Σ	implies "the combined effect of"
$G_{k,j}$	are the characteristic values of the permanent actions
A_d	is the design value of an accidental action
$Q_{k,1}$	is the characteristic value of the leading variable actions
$Q_{k,i}$	are the characteristic values of the other variable actions
$\psi_{1,1}$	is the factor for the frequent value of the leading variable action $Q_{k,1}$ (see Table 3.1)
$\psi_{2,i}$	is the factor for the quasi-permanent value of the i-th variable action $Q_{k,i}$ (see Table 3.1).

The values for ψ_1 and ψ_2 factors for buildings are given in Table 3.1. The values presented are taken from Table NA.A1.1 of the UK National Annex to BS EN 1990.

The accidental combination of actions is used when verifying the robustness of a structure, particularly with the notional removal and key element methods. The ψ factors are used in the calculation of the required tying resistance of connections for effective horizontal ties, see Section 6.2. The application of the combination of actions for the accidental design situation is explained in Section 7.

Table 3.1
Values of ψ_1 and ψ_2 factors for buildings (from the UK NA to BS EN 1990)

ACTION	ψ_1	ψ_2
Imposed loads in buildings (see BS EN 1991-1-1)		
Category A: domestic, residential areas	0.5	0.3
Category B: office areas	0.5	0.3
Category C: congregation areas	0.7	0.6
Category D: shopping areas	0.7	0.6
Category E: storage areas	0.9	0.8
Category H: roofs*	0	0
Snow loads on buildings (see EN 1991-3)		
For sites located at altitude H $>$ 1000 m a.s.l.	0.5	0.2
For sites located at altitude H $\leq$ 1000 m a.s.l.	0.2	0
Wind loads on buildings (see EN 1991-1-4)	0.2	0
Temperature (non-fire) in buildings (see EN 1991-1-5)	0.2	0

* On roofs, imposed loads should not be combined with either wind loads or snow loads.

3.2 BS EN 1993-1-1

BS EN 1993 is the Eurocode for design of steel structures: BS EN 1993-1-1 gives generic design rules for steel structures and specific guidance for structural steelwork used in buildings. BS EN 1993-1-1 gives no specific guidance for the design of steel buildings for structural robustness. Although BS EN 1993-1-1, 2.1.3 is entitled 'Design working life, durability and robustness' and states that steel structures shall be designed for accidental actions, it simply refers the reader to BS EN 1991-1-7 [1].

The absence of direct useful guidance contrasts with the more detailed provisions in BS 5950-1. Eurocodes are generally less specific and give general principles.

3.3 BS EN 1991-1-7

The requirement to design and construct buildings to have robustness and avoid disproportionate collapse under accidental design situations is established from BS EN 1990 (as described in Section 3.1). Details of how the requirement should be met are given in BS EN 1991-1-7.

3.3.1 Scope

BS EN 1991-1-7 provides strategies and rules for safeguarding buildings and other civil engineering works against accidental actions. However, the scope of this publication is limited to buildings.

Localised failure due to accidental actions can be acceptable provided that: it will not endanger the stability of the whole structure; the overall load-bearing resistance of the structure is maintained; the necessary emergency measures are able to happen.

The minimum period that most buildings need to survive following an accident should be that period needed to facilitate the safe evacuation and rescue of personnel from the building and its surroundings (i.e. the emergency measures). Longer periods of survival might be required for buildings used for handling hazardous materials, provision of essential services, or for national security reasons.

BS EN 1991-1-7 does not specifically deal with accidental actions caused by external explosions and terrorist activities, or the residual stability of buildings or other civil engineering works damaged by seismic action or fire.

If buildings are required to be designed to resist external explosions, warfare activities or terrorist activities these are additional design requirements outside the scope of BS EN 1991-1-7 and of this publication. SCI Publication P244 [16] *Protection of buildings against explosions* provides general guidance on the protection of commercial property and personnel from the effects of explosions caused by the detonation of high explosives.

Part 1 of BS EN 1998 [17] *Design of structures for earthquake resistance* provides general rules on seismic actions and rules for buildings. BS EN 1993-1-2 [18] *Eurocode 3, Design* *of steel structures, General rules, Structural fire design* provides general rules for structural fire design of steel buildings.

3.3.2 Design strategies

Two generic types of strategy for designing structures for accidental actions are provided in BS EN 1991-1-7:

a. Strategies based on identified accidental actions.
b. Strategies based on unidentified accidental actions.

These strategies are illustrated in Figure 3.1 of BS EN 1991-1-7, reproduced here in Figure 3.1.

BS EN 1991-1-7, 3.2 and 3.3 outline the strategies for identified accidental actions and for limiting the extent of localised failure, respectively.

Strategies based on unidentified accidental actions cover a wide range of possible events and are related to strategies based on limiting the extent of localised failure. The adoption of strategies for limiting the extent of localised failure might provide adequate robustness against those accidental actions not specifically covered by BS EN 1991-1-7 such as external explosions and terrorist activities, or any other action resulting from an unspecified cause. However, the adequacy of the robustness will be greatly dependant on the accidental action that is experienced.

Strategies based on identified accidental actions are naturally more specific. However, depending on the exact nature of the strategy, the structure might also possess adequate robustness against some unidentified actions.

For the majority of steel framed buildings, the potential accidental actions will remain unidentified and therefore the approach of limiting the extent of localised failure is likely to be the general strategy adopted.

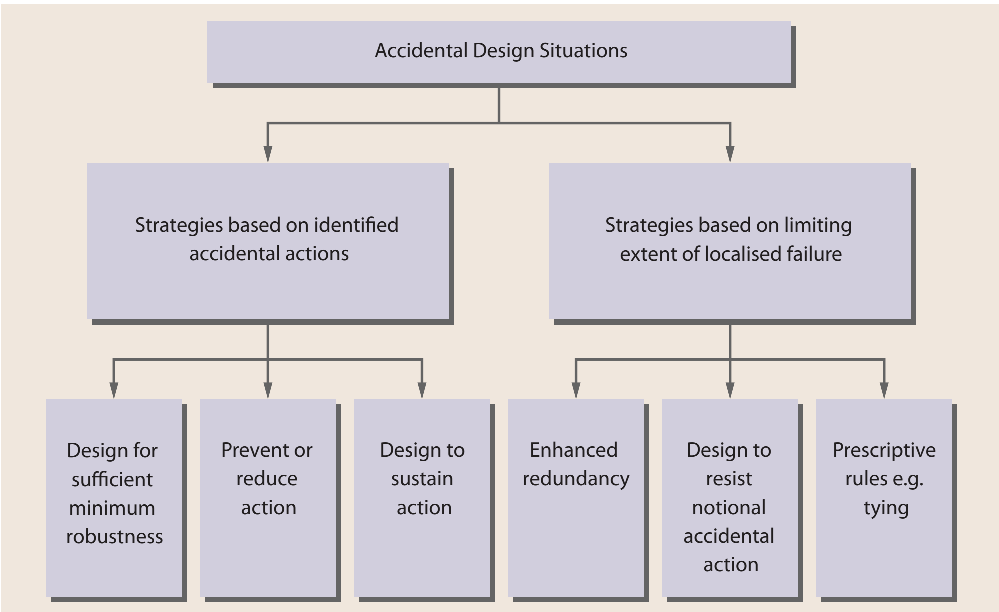

Figure 3.1 Strategies for accidental design situations

Strategies for identified accidental actions

The strategy for identified accidental actions is set out in BS EN 1991-1-7, 3.2. The main features are:

> *A localised failure due to accidental actions may be acceptable, provided it will not endanger the stability of the whole structure and that the overall load-bearing capacity of the structure is maintained and allows the necessary emergency measures to be taken.*
>
> *Measures should be taken to mitigate the risk of accidental actions and these measures should include, as appropriate, one or more of the following strategies:*
>
> - *preventing the action from occurring…*
> - *protecting the structure against the effects of the accidental action…*
> - *ensuring that the structure has sufficient robustness, by adopting one or more of the following approaches:*
>   - *by designing certain components of the structure upon which stability depends as key elements…*
>   - *designing structural members, and selecting materials, to have sufficient ductility, capable of absorbing significant strain energy without rupture.*
>   - *incorporating sufficient redundancy in the structure to facilitate the transfer of actions to alternative load paths following an accidental event.*

Strategies for limiting the extent of localised failure

Where accidental actions are unidentified, strategies for limiting the extent of localised failure may be applied. BS EN 1991-1-7, 3.3 (1)P sets out the following principle:

> *In the design, the potential failure of the structure arising from an unspecified cause shall be mitigated.*
>
> *The mitigation should be reached by adopting one or more of the following approaches:*
>
> - *Designing key elements, on which the stability of the structure depends, to sustain the effects of a model of accidental action Ad;*
> - *Designing the structure so that in the event of a localised failure (e.g. failure of a single member) the stability of the whole structure or of a significant part of it would not be endangered;*
> - *Applying prescriptive design/detailing rules that provide acceptable robustness for the structure (e.g. three dimensional tying for additional integrity, or a minimum level of ductility of structural members subject to impact).*

Examples relating to the use of the approaches for buildings are given in Annex A of BS EN 1991-1-7 (see Section 3.3.3).

Consequences classes

BS EN 1991-1-7, 3.4 permits the strategies for accidental design situations to be based on consequences classes set out in BS EN 1990. The guidance given is summarised in Table 3.2.

The classification of buildings is described in detail in Section 4.

CONSEQUENCES CLASS	CONSIDERATIONS FOR ACCIDENTAL DESIGN SITUATIONS
CC1	No specific consideration is necessary for accidental actions except to ensure that the robustness and stability rules given in EN 1990 to EN 1999, as applicable, are met.
CC2	Depending upon the specific circumstances of the structure, a simplified analysis by static equivalent action models may be adopted or prescriptive design/detailing rules may be applied.
CC3	An examination of the specific case should be carried out to determine the level of reliability and the depth of structural analyses required. This might require a risk analysis to be carried out and the use of refined methods such as dynamic analyses, non-linear models and interaction between the load and the structure.

Table 3.2 Consequences class design strategies

3.3.3 Annex A

Annex A of BS EN 1991-1-7 presents more detailed guidance than that given in Section 3 of BS EN 1991-1-7. Annex A is an 'informative' annex rather than 'normative' meaning that it is for information and the guidance does not have to be followed. However, the UK National Annex decision on the status of the informative Annex A is to recommend the application of Annex A by stating that:

> *Guidance in Annex A of BS EN 1991-1-7:2006 should be used in the absence of specific requirements in BS EN 1992-1-1 to BS EN 1996-1-1 and BS EN 1999-1-1 and their National Annexes.*

There are no specific requirements given in BS EN 1993-1-1. Therefore, it is recommended that the guidance in Annex A of BS EN 1991-1-7 should be followed for steel framed buildings.

Compliance with BS EN 1991-1-7, Annex A can be used to demonstrate compliance with the requirement to ensure the avoidance of disproportionate collapse in the UK Building Regulations.

The scope of BS EN 1991-1-7, Annex A is to give rules and methods for designing buildings to sustain an extent of localised failure from an unspecified cause without disproportionate collapse. While other approaches might be equally valid, adoption of the Annex A strategy is likely to ensure that a building is sufficiently robust to sustain a limited extent of damage or failure without excessive collapse.

The main topics for which Annex A provides guidance are:

- Consequences classes for buildings.
- Recommended strategies.
- Effective horizontal ties.
- Effective vertical ties.
- Key elements.

Explanation and application of the guidance of Annex A to hot-rolled steel framed buildings is provided in the following Sections of this publication.

It is important to understand that the rules in Annex A are best considered as prescriptive rules intended to strike a balance between cost and safety and which experience suggests produce structures that generally perform adequately in extreme circumstances. The rules are not meant to be fully described systems of structural mechanics. It is important that designers are not excessively theoretical about providing a solution to robustness design.

3.4 Robustness strategies

The recommended strategies to provide an acceptable level of robustness for each consequences class are presented in BS EN 1991-1-7, Annex A, A.4. The structural requirements are progressively more stringent from Class 1 through to Class 3, which reflects the increase in consequences if collapse were to occur.

Adoption of the recommended strategies given in BS EN 1991-1-7, A.4 (1) is intended to provide a building with an acceptable level of robustness to sustain localised failure without a disproportionate level of collapse.

Sections 5 to 8 of this publication describe the structural requirements of the BS EN 1991-1-7 recommended robustness strategies for each consequences class. Sections 5 to 8 also advise of additional structural provisions that can be applied together with the recommendations of BS EN 1991-1-7.


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BUILDING CLASSIFICATION

4.1 Consequences classes

Table A.1 in Annex A of BS EN 1991-1-7 provides a method to categorise buildings in four consequences classes. The building categorisation considers the building type, occupancy and size.

The method of building classification given in Annex A of BS EN 1991-1-7 is very similar to that given in Approved Document A guidance for the England and Wales Building Regulations, except that Annex A uses Classes 2a and 2b whereas the notation 2A and 2B is used in Approved Document A.

In BS EN 1990 and BS EN 1991-1-7, 3.4 there are three consequences classes. In Annex A of BS EN 1991-1-7 consequences class 2 has been subdivided into CC2a (Lower risk group) and CC2b (Upper risk group), creating a total of four classes. The categorisation from Annex A of BS EN 1991-1-7 is shown in Table 4.1.

The building classification is a simplification of a complex risk-based building classification system. The classes are only partly related to the building size, the other main factor is the building use which takes account of socio-economic factors. Hence, hospitals and schools, for example, generally have a higher classification than other buildings of a similar size. The risk-based approach calculates a risk factor for each type of building based on the following variables:

- The number of people at risk.
- The location of the structure and its height.
- The perception in society of damage to the structure.
- The type of load and likelihood that the load will occur at the same time as a large number of people being present within or near the structure.
- The structural type and nature of the material.

Further information on the classification process is provided in Reference 19.

4.2 Practical cases

In practice, many buildings will not fall simply into one of the descriptions given in Table 4.1. There are many reasons why this could be the case, for example mixed use, basements and varying number of storeys.

CONSEQUENCES CLASS	BUILDING TYPE AND OCCUPANCY
1 Low consequences of failure	Single occupancy houses not exceeding 4 storeys. Agricultural buildings. Buildings into which people rarely go, provided no part of the building is closer to another building, or area where people do go, than a distance of 1.5 times the building height.
2a (Lower risk group) Medium consequences of failure	5 storey single occupancy houses. Hotels not exceeding 4 storeys. Flats, apartments and other residential buildings not exceeding 4 storeys. Offices not exceeding 4 storeys. Industrial buildings not exceeding 3 storeys. Retailing premises not exceeding 3 storeys of less than 1000 m^2 floor area in each storey. Single storey educational buildings. All buildings not exceeding two storeys to which the public are admitted and which contain floor areas not exceeding 2000 m^2 at each storey.
2b (Upper risk group) Medium consequences of failure	Hotels, flats, apartments and other residential buildings greater than 4 storeys but not exceeding 15 storeys. Educational buildings greater than single storey but not exceeding 15 storeys. Retailing premises greater than 3 storeys but not exceeding 15 storeys. Hospitals not exceeding 3 storeys. Offices greater than 4 storeys but not exceeding 15 storeys. All buildings to which the public are admitted and which contain floor areas exceeding 2000 m^2 but not exceeding 5000 m^2 at each storey. Car parking not exceeding 6 storeys.
3 High consequences of failure	All buildings defined above as Class 2 Lower and Upper Consequences Class that exceed the limits on area and number of storeys. All buildings to which members of the public are admitted in significant numbers. Stadia accommodating more than 5000 spectators. Buildings containing hazardous substances and /or processes.

Table 4.1 Categorisation of consequences classes

Notes:

1. For buildings intended for more than one type of use, the “consequences class” should be that relating to the most onerous type.
2. In determining the number of storeys, basement storeys may be excluded, provided that such basement storeys fulfil the requirements of “Consequences Class 2b Upper Risk Group”.

Additional guidance on classification of buildings has been published in References 20, 21 and 24. The additional guidance was originally produced for use with Approved Document A, but since the classification system in Annex A of BS EN 1991-1-7 is almost exactly the same as in Approved Document A the additional guidance is equally applicable for use with BS EN 1991-1-7.

The only difference between the building classification system in Approved Document A and in BS EN 1991-1-7 is the floor area limit for Class 2a retailing premises. In Approved Document A, the definition of Class 2a retailing premises is *"premises not exceeding 3 storeys of less than 2000 m^2 floor area in each storey"* whereas in BS EN 1991-1-7 the definition is *"premises not exceeding 3 storeys of less than 1000 m^2 floor area in each storey"*.

Guidance on building classification issues is given in Sections 4.2.1 to 4.2.6. It is important that guidance is not followed without also judging each case on its merits.

4.2.1 Mezzanine floors

The classification in Annex A of BS EN 1991-1-7 requires the number of storeys in the building to be counted; whether a mezzanine floor should be counted as a storey or not depends on its size and its use. Each situation needs to be judged on its merits. However, as an approximate guide, a mezzanine floor should be considered as a storey if it is greater than 20% of the building footprint. If personnel are not accessing the mezzanine floor daily then it might be reasonable to increase the limit. Guidance on the design of mezzanine floors for lateral stability is provided in Advisory Desk Note AD267 [22].

4.2.2 Habitable roof spaces

Habitable areas of roof space should generally be counted as a storey, irrespective of the slope of the roof. Roof spaces in residential buildings that are used to house only plant and water tanks need not be considered as a storey.

For loft conversions, a case can be developed for not counting the loft accommodation as an additional storey in circumstances where:

- The occupancy of the building has not increased significantly (e.g. where the loft conversion provides additional space for the current inhabitants rather than as a self-contained apartment).
- The existing line of the roof is maintained, except perhaps for the addition of dormer windows on one elevation.

These circumstances should be confirmed with the building control authority.

4.2.3 Buildings with a varying number of storeys

Buildings with a varying numbers of storeys that fall into more than one consequences class should be classified as the more onerous class. The robustness measures for the more onerous class should continue until a structural discontinuity (such as a movement joint) is reached, provided that the building either side of the movement joint is structurally independent and robust in its own right.

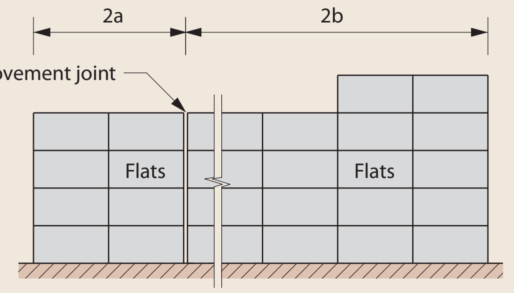


Figure 4.1 Classification of 4 and 5 storey flats

Example

Figure 4.1 shows a block of flats partly of 4 storeys and partly of 5 storeys. Class 2b robustness strategies should be applied to the 5 storey areas and extending to a suitable structural discontinuity in the 4 storey area and Class 2a robustness strategies may be applied to the remaining 4 storey area.

4.2.4 Mixed use buildings

For buildings intended for more than one type of use the class should be that pertaining to the most onerous type. Where different occupancies are in horizontally adjacent parts of the same building, the same approach to consequences classes may be adopted as described in Section 4.2.3 for buildings with varying numbers of storeys, i.e. the consequences class for the more onerous class should continue horizontally until a structural discontinuity (such as a movement joint) is reached.

The following examples illustrate the classification of mixed use buildings.

Examples

Figure 4.2 shows 2 storeys of flats over 1 storey of retailing premises. This case should be considered as 3 storeys of retailing premises. Therefore, apply Class 2a robustness strategies to the whole building, or apply Class 2b robustness measures to the whole building if floor area of retailing premises is 1000 m^2 or more (per storey).

(Note: The 1000 m^2 limit used here is taken from BS EN 1991-1-7, Annex A, the equivalent limit in Approved Document A is 2000 m^2.)

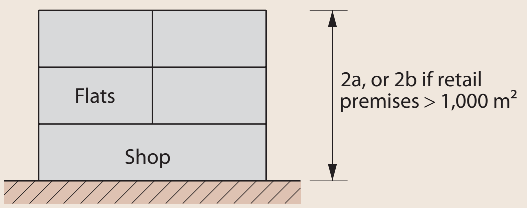


Figure 4.2 Classification of 2 storey flats over 1 storey retail

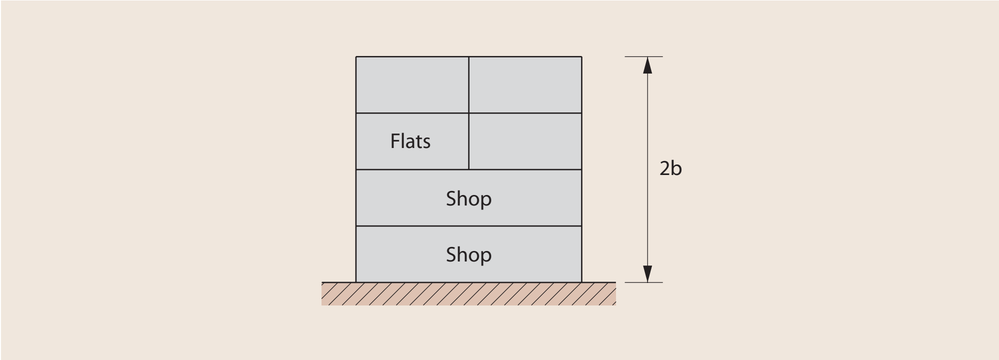


Figure 4.3 Classification of 2 storey flats over 2 storey retail

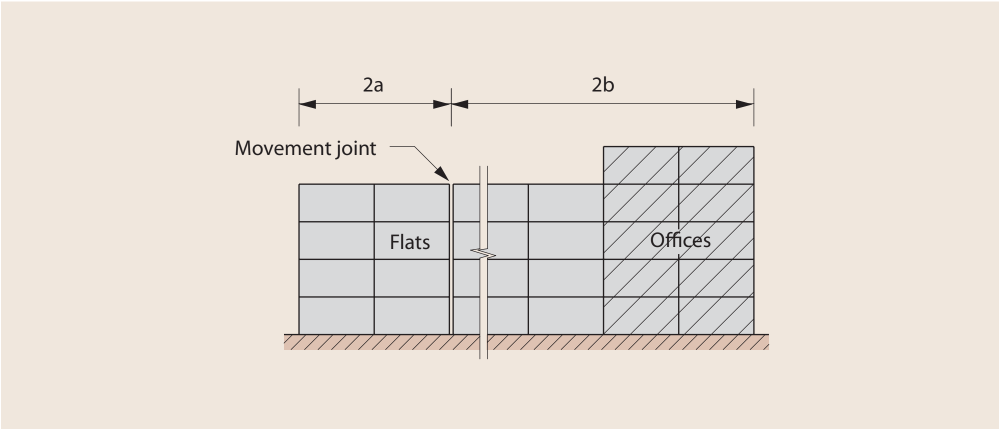


Figure 4.4 Classification of 4 storey flats attached to 5 storey offices

Figure 4.3 shows 2 storeys of flats over 2 storeys of retailing premises. This case should be taken as 4 storeys of retailing premises. Therefore, apply Class 2b robustness strategies to the whole building.

Figure 4.4 shows 4 storeys of flats adjacent to 5 storeys of offices. Class 2b robustness strategies should be applied to the 5 storey office area and extending to a suitable structural discontinuity in the 4 storey residential area and Class 2a robustness strategies should be applied to the remaining 4 storey residential area.

4.2.5 Buildings with basements

To qualify as a basement storey for the purpose of building classification, the external ground level should be at least 1.2 m above the top surface of the basement floor for a minimum of 50% of the plan area of the building (see Reference 20).

The robustness strategies that are required to be applied to the part of the building above the basement depend on the total number of storeys and the robustness strategies applied to the basement storey.

In determining the number of storeys for classification, basement storeys may be excluded if they fulfil the robustness requirements of Class 2b buildings. Otherwise, the basement storeys must be included in determining the number of storeys for building classification.

The basement can be for habitable accommodation or parking. The following examples illustrate the appropriate classification to be applied.

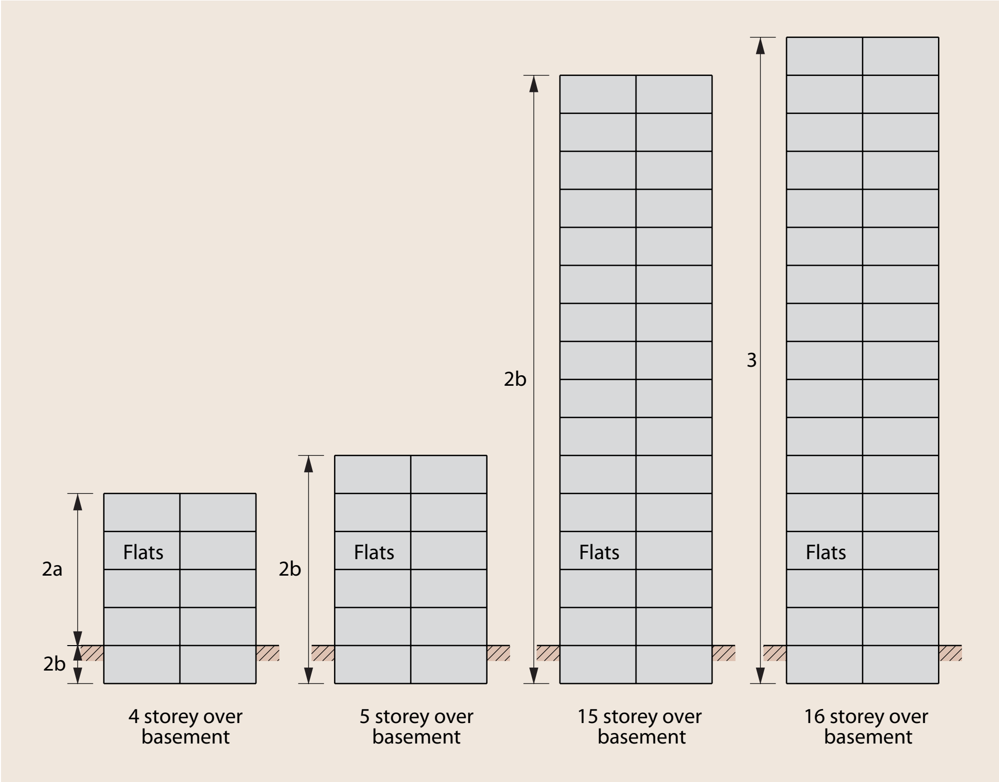


Figure 4.5 Classification of flats over basements

Examples

Figure 4.5 shows buildings with storeys of flats above basements and the class of robustness strategy to be applied.

4.2.6 Ground floor storey

In determining the number of storeys for classification, the ground floor storey may generally be excluded if all the structural elements of the ground floor storey, and their connections, are designed as key elements (see Reference 20). Section 7.7 gives guidance on the design of key elements.

Where used for parking, the ground floor storey may only be excluded (as described above) if all of the following conditions apply:

i. Parking is exclusively for users of the building.
ii. The ground floor storey must not be accessible to or contain a right of way for the general public.
iii. All the structural elements of the ground floor storey, and their connections, are designed as key elements.

The following examples illustrate the appropriate classification to be applied.

Examples

Figure 4.6 shows blocks of flats and the class of robustness strategy to be applied from using the option to design the ground floor storey members as key elements.

Figure 4.6 Classification options for flats

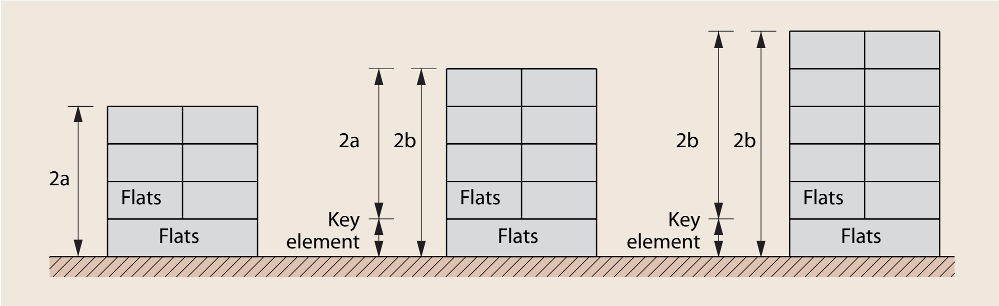


Figure 4.7 Classification of flats with ground floor car park

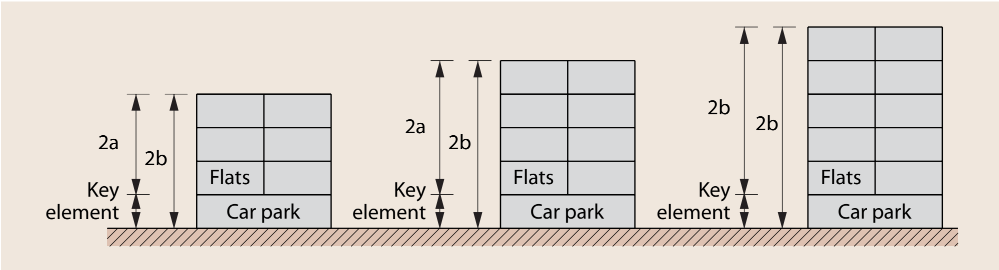


Figure 4.7 shows blocks of flats above a ground floor storey used as a car park and the consequences class to be applied for robustness from using the option to design the ground floor storey members as key elements. In this example it is assumed that the parking is exclusively for users of the building and that the ground floor storey is not be accessible to and does not contain a right of way for the general public.

Reference 24 provides additional guidance on the classification of buildings where a mix of structural materials has been used, particularly where a ground floor podium has been used.

4.3 Conversions, alterations and extensions

Buildings that undergo conversions, alterations or extensions (or a combination of these) can have changed consequences class as a result of the work. Where the work has caused the building to be categorised into a more onerous class (e.g. due to a rooftop extension) the potential requirement to apply the robustness strategy appropriate to the more onerous class throughout the building may be economically prohibitive. The structural requirements in such cases should be clarified and agreed with the local building control body. One solution might be to use the ‘Camden Ruling’, which has been used in the past in the UK [23]. The Camden Ruling allows the designer to adopt a solution that demonstrates that any damage occurring within the storeys of a rooftop extension would be contained by the floor forming the roof of the original building i.e. the roof of the original building can support the debris loading of the rooftop extension. If this can be achieved, the alteration appears not to change the risk to the occupants of the original lower floors. However, the Camden Ruling method is not universally accepted and can be considered as controversial because additional storeys will almost certainly increase the risk on the lower storeys. Further discussion on the method is given in Reference 24.

BUILDINGS IN CONSEQUENCES CLASS 1

5.1 Structural requirements

5.1.1 Robustness strategy

The recommended strategy in BS EN 1991-1-7, Annex A, A.4 for Consequences Class 1 buildings states:

> *Provided a building has been designed and constructed in accordance with the rules given in EN 1990 to EN 1999 for satisfying stability in normal use, no further specific consideration is necessary with regard to accidental actions from unidentified causes.*

Hence, for steel-framed buildings designed in accordance with the rules given in BS EN 1993 no additional rules need to be applied for compliance with BS EN 1991-1-7 for the consideration of avoidance of disproportionate collapse.

5.1.2 Additional structural provisions

In addition to adopting the above strategy, it is recommended that a minimum level of horizontal tying is provided within the frame. The recommended minimum level of horizontal tying is that all floor beam-to-column connections are designed to be capable of sustaining a design tensile force of 75 kN.

5.2 Minimum horizontal tying

5.2.1 Benefits of providing a minimum level of horizontal tying

The purpose of providing a minimum level of horizontal tying is to ensure that beam-to-column connections are not impaired by relatively small horizontal or upward actions applied to the beams that could cause beams to collapse onto floors below.

If only gravity and horizontal actions were to be considered, it would be theoretically possible to design beam-to-column connections within a braced frame that only have a vertical shear resistance, e.g. a beam seated on a bearing block welded to a column. These types of connection are not normally used in the UK, since the beam could easily be dislodged. It is good engineering practise to detail all connections to have a minimum level of horizontal resistance.

5.2.2 Design rules

For a minimum level of horizontal tying the following design rules are recommended:

a. All floor beam-to-column connections should be designed to be capable of sustaining a design tensile force of 75 kN.
b. The 75 kN tie force need not be combined with the effects of any other actions.

The recommendation should also be applied at roof level, except where the steelwork only supports roof cladding that weighs not more than 0.7 kN/m^2 and carries only imposed roof loads and wind loads.

It is not necessary that beam-to-beam connections are designed for a specific tying force in order to satisfy a robustness strategy for Class 1 buildings as the minimum tying recommendation applies only to beam-to-column connections. Hence, it is not necessary to design secondary beams as ties, as shown in Figure 5.1.

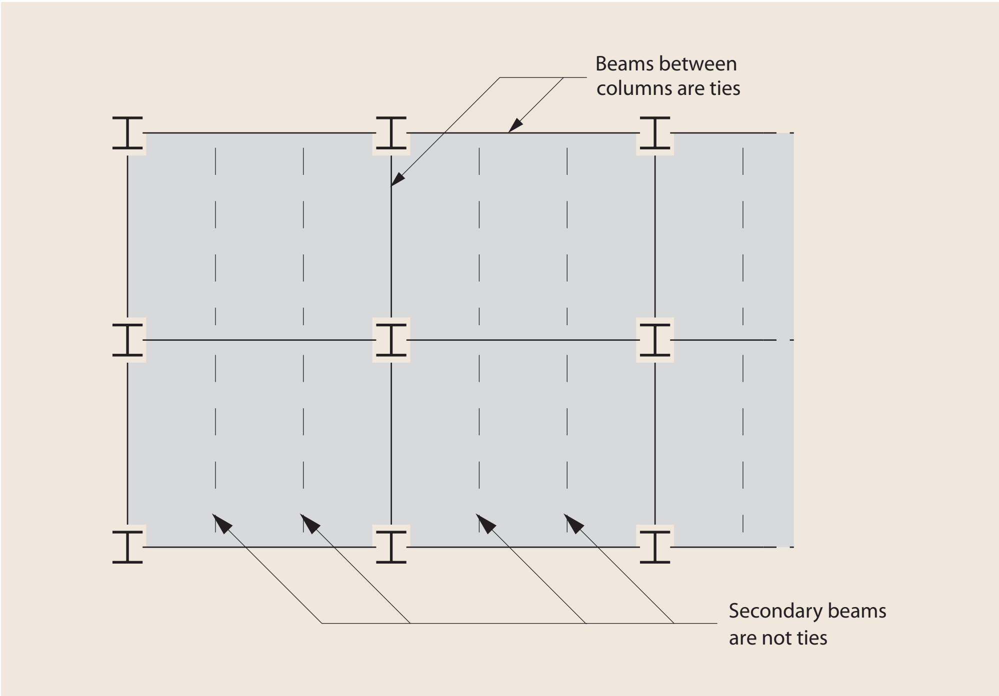


Figure 5.1 Ties recommended in Class 1 buildings

5.3 Practical application of design rules

In order to design columns as restrained in position for each axis at each floor and roof level, the columns should generally be connected to beams, approximately at right angles, at each principal floor level.

Providing resistance to the 75 kN tying force will generally not be an onerous requirement for connections in hot-rolled steel frames. All standard beam-to-column connections for simply supported beams have a tying resistance that exceeds 75 kN. Design tables for standard simple connection types (see Figure 5.2) are provided in SCI publication P358 [25].

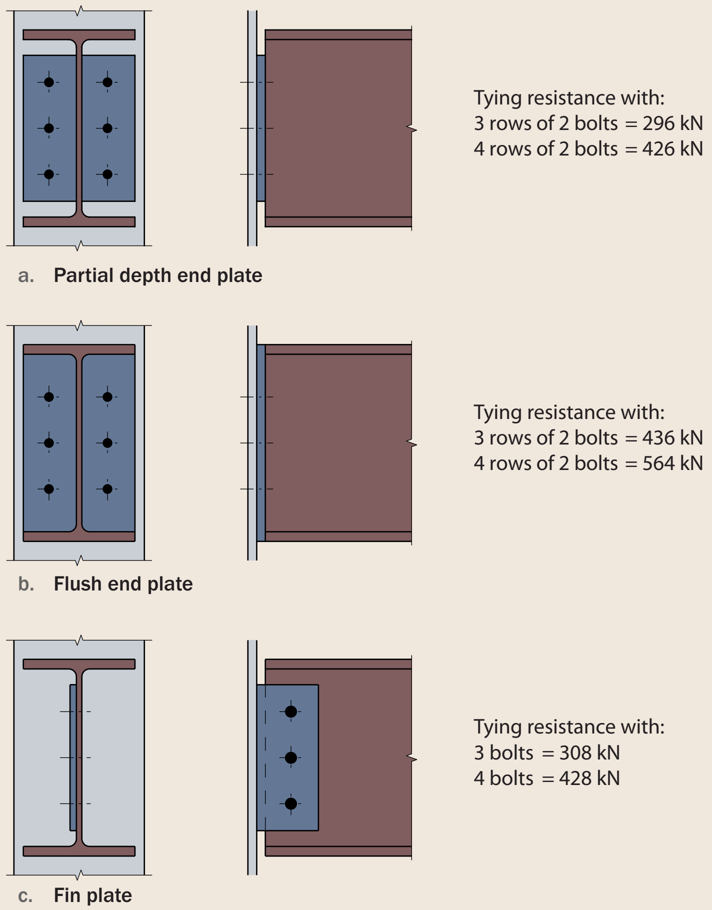
BS EN 1993-1-8 [26] does not give any guidance on the tying resistance of connections. Large strains and large deformations are acceptable in the accidental design situation. Therefore, for the calculation of connection tying resistance, SN015 [27] recommends that ultimate tensile strengths (f_u) be used and the partial factor for tying $\gamma_{M,u}$ be taken as 1.1. Examples showing the calculation of connection resistances are included in SCI publication P364 [28].

Design resistances for M16 and M20 class 8.8 bolts in S275 steel are given in Table 5.1. Any reasonable connection with at least two bolts will provide a tying resistance in excess of the 75 kN requirement.

Table 5.1
Design resistances for class 8.8 bolts in S275 steel

BOLT DIAMETER	TENSION RESISTANCE	SINGLE SHEAR RESISTANCE	BEARING RESISTANCE (8 MM PLATE)
M16	90.4 kN	60.3 kN	54.5 kN
M20	141 kN	94.1 kN	67.4 kN

Note: The above values have been calculated in accordance with BS EN 1993-1-8 [26].



Note: Values for 3 rows of bolts are for a 356 x 171 x 67 S275 UKB
Values for 3 rows of bolts are for a 356 x 171 x 67 S275 UKB

Figure 5.2
Types of simple connection with typical tying resistance

BUILDINGS IN CONSEQUENCES CLASS 2A

6.1 Structural requirements

6.1.1 Robustness strategy

The recommended strategy in BS EN 1991-1-7, Annex A, A.4 for Consequences Class 2a buildings states:

> *In addition to the recommended strategies for Consequences Class 1, the provision of effective horizontal ties, or effective anchorage of suspended floors to walls, as defined in A.5.1 and A.5.2 respectively for framed and load-bearing wall construction should be provided.*
>
> *Note 1: Details of effective anchorage may be given in the National Annex.*

Load-bearing wall construction is outside the scope of this publication and the requirement for effective anchorage is not discussed. The requirements of effective horizontal ties for framed structures as set out in A.5.1 of BS EN 1991-1-7, are described in Section 6.2.

Approved Document A

The wording of the recommended strategy is similar to the guidance given in Approved Document A (2004) for Class 2A buildings. However, it should be noted that interpretation of what constitutes 'effective horizontal ties' is considerably different in magnitude. The Approved Document A (2004) interpretation is that 75 kN constitutes an effective horizontal tie for hot-rolled steel frame construction. In BS EN 1991-1-7, 75 kN is a minimum tensile load that the tie should sustain. The actual tie force is calculated and in many cases will be significantly greater than 75 kN (see Section 6.3.8).

6.1.2 Additional structural provisions

In addition to the robustness strategy from BS EN 1991-1-7, bearing details for floor, roof and stair units should conform to BS EN 1992 and make due allowance for construction, fabrication and manufacturing tolerances.

6.2 Horizontal ties

6.2.1 Benefits of providing horizontal ties

Horizontal tying can be beneficial to a structure in an accidental action situation by:

- enabling catenary action to develop;
- holding columns in place.

These two potential roles of horizontal tying are discussed below.

Catenary action

The principle of providing horizontal ties notionally allows for beam members to support loads by forming catenaries over damaged areas of structure. The provision of horizontal ties, designed to the Eurocode rules, has no complementary requirements relating to joint ductility or joint rotation capacity. The robustness rules are not meant to fully describe systems of structural mechanics but are considered as rules intended to produce structures that perform adequately in accidental circumstances. Nonetheless, applying the rules contributes towards support over damaged areas of structure where the support provided by a column has been lost, as shown in Figure 6.1.

Figure 6.1 shows forces redistributed vertically as well as horizontally. However, for Consequences Class 2a buildings, vertical tying is not a requirement of BS EN 1991-1-7 or Approved Document A [12]. Therefore, Figure 6.1 does not show forces being redistributed vertically past the first column splice above the damaged column section. Column members are generally spliced every second or third storey so, depending on the location of the damage, there will usually be some upward transfer of forces at least as far as the first column splice.

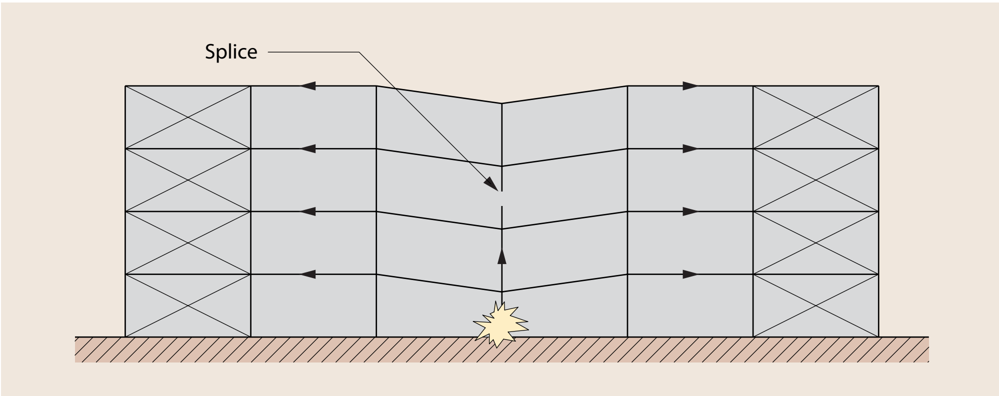


Figure 6.1 Catenary action with horizontal ties

Holding columns in place

Accidental actions can cause horizontal forces to act on column sections: ensuring that beam-to-column connections have tying resistance helps to hold the column in place and therefore that it can continue to support vertical loads, as shown in Figure 6.2. The accidental action shown in Figure 6.2 is depicted as an internal blast but the principle applies to any accidental action that can cause horizontal forces. Holding columns in

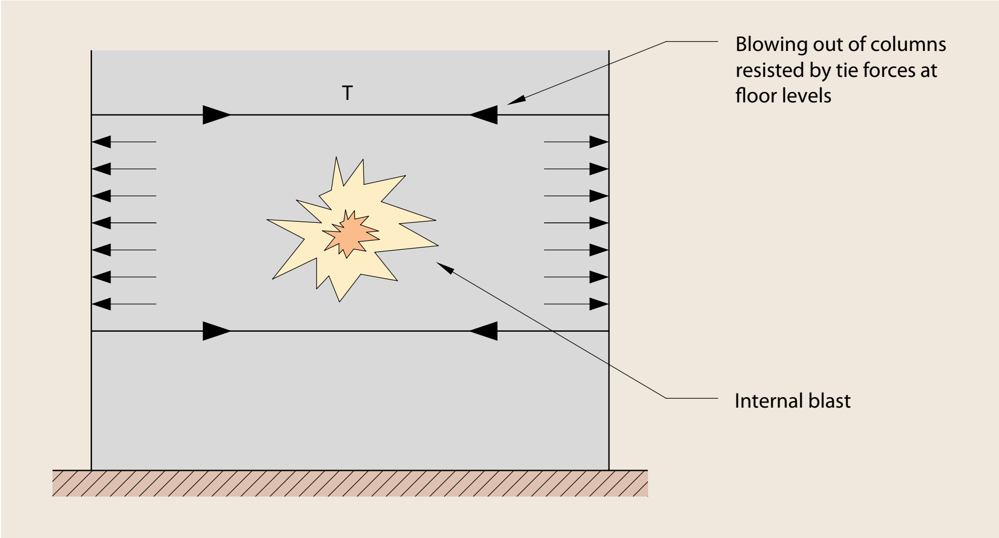


Figure 6.2 Columns held in position with horizontal ties

place also, importantly, helps to prevent floor units falling due to the spread of beams that could occur if columns were not held in position.

6.2.2 Design forces

The requirements for effective horizontal ties, as defined in A.5.1 of BS EN 1991-1-7, are given below.

Each tie member, including its end connections, should be capable of sustaining a design tensile load of T_i for the accidental limit state in the case of internal ties, and T_p, in the case of perimeter ties. The magnitudes of T_i and T_p are calculated according to equations A.1 and A.2 from BS EN 1991-1-7, A.5.1, reproduced below:

$T_i \quad = 0.8(g_k + \psi q_k)\,s\,L$ or 75 kN, whichever is the greater

$T_p \quad = 0.4(g_k + \psi q_k)\,s\,L$ or 75 kN, whichever is the greater

where:

g_k is the permanent action
q_k is the variable action
s is the spacing of the ties
L is the span of the tie
ψ is the relevant factor in the expression for combination of action effects for the accidental design situation (i.e. ψ_1 or ψ_2 in accordance with expression (6.11b) of EN 1990).

Note: The reference to expression (6.11b) of EN 1990 in the definition of ψ is specific to the ψ factor. Expression (6.11b) of EN 1990 is not used to determine the tie force.

Note that the permanent action of cladding loads does not need to be included in the expression for T_p as the cladding load would no longer be present on the member in this design situation.

The UK National Annex to BS EN 1990 specifies that ψ_1 should be used for combinations with accidental actions (see 3.1.3). Values for ψ_1, taken from Table NA. 1.1, are given in Table 3.1. Values of ψ_1 for floor loads in buildings vary from zero for roofs to 0.9 for storage areas. For the common situations of office areas and residential areas, ψ_1 should be taken as 0.5.

It is not necessary for the tie force to be combined with any other loads. Therefore, the connection does not need to be designed to resist a shear load and a tensile load from the ties simultaneously.

6.2.3 Provision of ties

The horizontal ties should be:

a. Provided around the perimeter of each floor.
b. Provided around the perimeter of the roof level.
c. Provided internally in two right angle directions to tie the columns securely to the structure of the building.
d. In continuous lines.
e. For perimeter ties, arranged as closely as practicable to the edges of floors.
f. For ties intended to be on column lines, arranged as closely as practicable to the lines of columns.
g. Arranged so that at least 30% of the ties are located within the close vicinity of the grid lines of the columns.

Figure 6.3 shows the location of horizontal ties in a floor of a Class 2a building.

The horizontal ties can comprise the following elements or a combination of them:

a. Rolled steel sections (i.e. floor beam members).
b. Steel bar reinforcement in composite steel/concrete floors.
c. Steel fabric (mesh) reinforcement in composite steel/concrete floors.
d. Profiled steel sheeting in composite steel/concrete floors.
e. Precast units, if effectively tied to the steel beams.

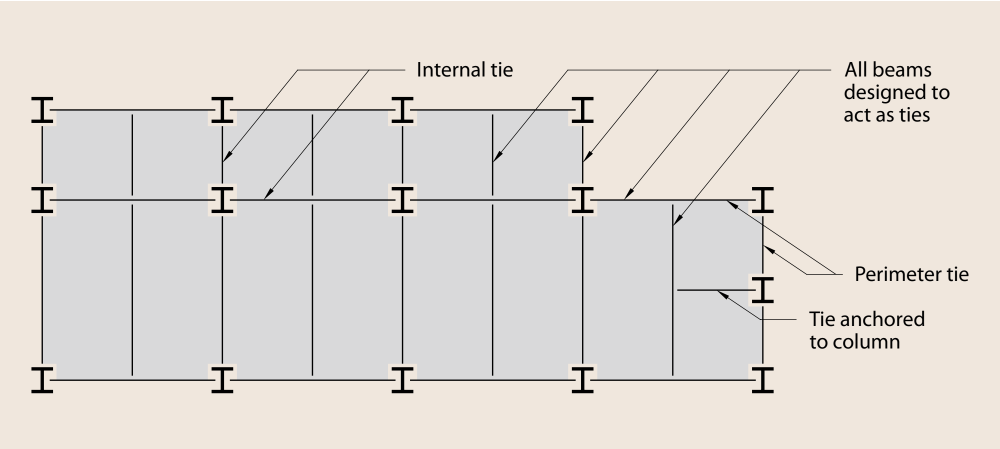


Figure 6.3 Location of horizontal ties in Class 2a buildings

Note: Items (b), (c) and (d) may only be utilised as ties if the composite steel/concrete floors are connected directly to the steel beams with shear connectors. Item (d) may only be utilised as ties acting in the same direction of the span of the profiled steel sheeting and where the sheeting is directly fixed to the supporting steel beam.

6.3 Practical application of design rules

6.3.1 Chasing loads

There is no requirement in building regulations or BS EN 1991-1-7 to 'chase' the tie forces around the structure. The requirement is only to design the member and its end connection for the tie force; the designer is not required to consider the consequences of the tie forces any further. This point is illustrated by reference to Figure 6.4 which shows a horizontal tie force acting on an external column. The external column section AB does not need to be designed for a lateral force.

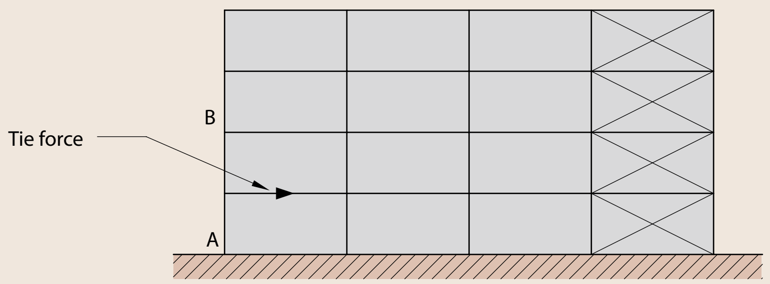


Figure 6.4 Tie force on external column

For a beam connected to a column web with an end plate connection, the column web should be checked to ensure that it can sustain the tying force (large deformations are acceptable) but the column section as a member does not need to be checked.

6.3.2 Unequal spans

As stated previously, providing horizontal ties notionally allows for beam members to support loads by forming catenaries. Where a line of ties consists of beams with different spans (as shown in Figure 6.5) the design tie forces will be different along the line of ties. This is at odds with the theoretical tensile force in a catenary which

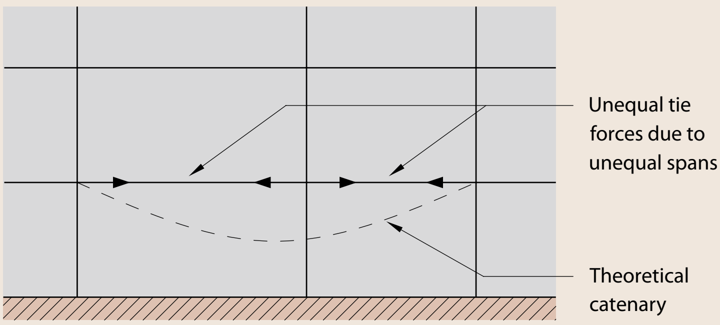


Figure 6.5 Tie force with unequal spans

is constant throughout its length. However, when considering unidentified accidental actions, the designer is not required to consider this theoretical inconsistency and no rationalisation of design tie forces along a line of ties is required. The same theoretical inconsistency can occur when beams are supporting different levels of load.

6.3.3 Continuous lines

BS EN 1991-1-7, A.5.1 states that the horizontal ties should be arranged in continuous lines. Where there are irregular column grids or beam arrangements resulting in an offset between lines of ties (as shown in Figure 6.6), the beams are not in continuous lines. The designer needs to be satisfied that the tie force can be transferred from one beam to the next. Alternative elements such as the floor slab may be used, or in extreme cases, an additional member can be added to transfer the tie force. Depending on the size of the tie force, and if the offset or discontinuity is small, it is possible to justify that the tie force can be transferred through bending and shear of connected members.

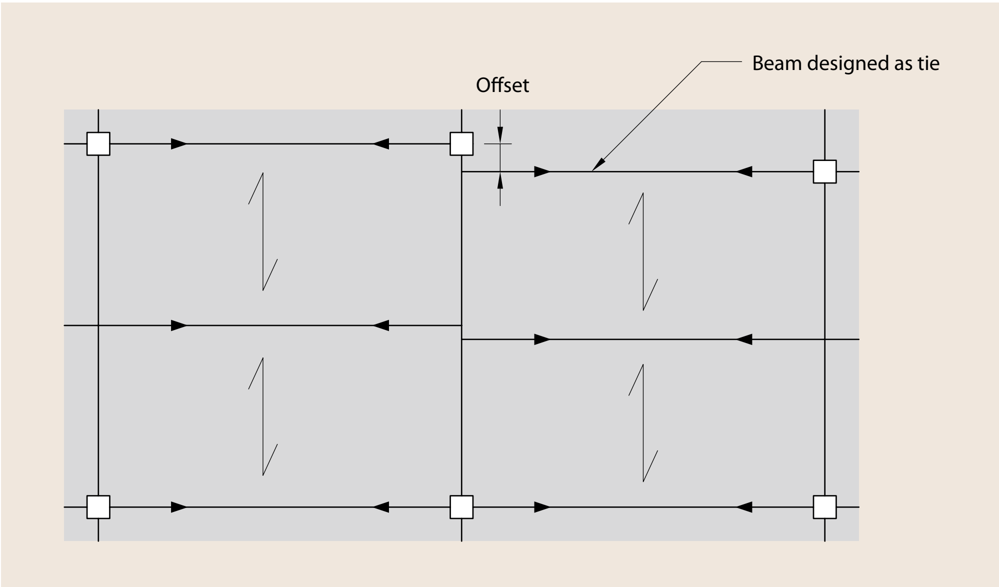


Figure 6.6 Discontinuity in line of ties

6.3.4 Voids

Another situation where it could be argued that the ties are not continuous is where a line of ties is interrupted due to a void in a building such as an atrium. In this case there is no requirement to make additional allowance for the void in the floor. The void or atrium should be treated in the same way as any other floor or roof edge in the building. Hence, horizontal ties should be provided around the perimeter of each floor and roof level, which includes the perimeter adjacent to voids in the floor such as atria.

6.3.5 Beams not on column lines

BS EN 1991-1-7 is ambiguous about whether beams that are not on column lines should be designed as ties or not. BS EN 1991-1-7 A.5.1 (1) states that horizontal ties should be provided as closely as practicable to lines of columns and at least 30% of the ties should be located within close vicinity of the grid lines of columns. In contrast, label (b) of Figure A.2 indicates that all beams should be designed to act as ties. Therefore, it is concluded that equations A.1 (or A.2) should be applied to all beams whether or not they are on the column lines. The expression “30% of the ties” is then interpreted to mean 30% of the total required tie resistance. Hence, the designer must also check that at least 30% of the required tie resistance is in close vicinity to the column line. No guidance is provided in BS EN 1991-1-7 as to the definition of close vicinity. For the purposes of this publication, close vicinity is taken to mean a quarter of the column spacing.

Figure 6.7 shows an arrangement of internal floor beams. Equation A.1 of BS EN 1991-1-7 A.5.1 should be applied to each beam in Figure 6.7 to determine the required minimum tying resistance (T_i) of the beam and its end connections. Depending on the loading and beam spacing, each beam can have a different minimum tying resistance. In addition to verifying that the member and its end connections can resist the calculated tying load, the designer must also verify that at least 30% of the tie resistance is within a distance of a quarter of the column spacing either side of the column line. Therefore, considering the beam on column line ‘B’ in Figure 6.7, the following requirement should be satisfied:

$$T_5 \qquad \geq 0.3\,(T_4 + T_5 + T_6)$$

Equivalent expressions should also be satisfied for beams on column lines ‘A’ and ‘C’.

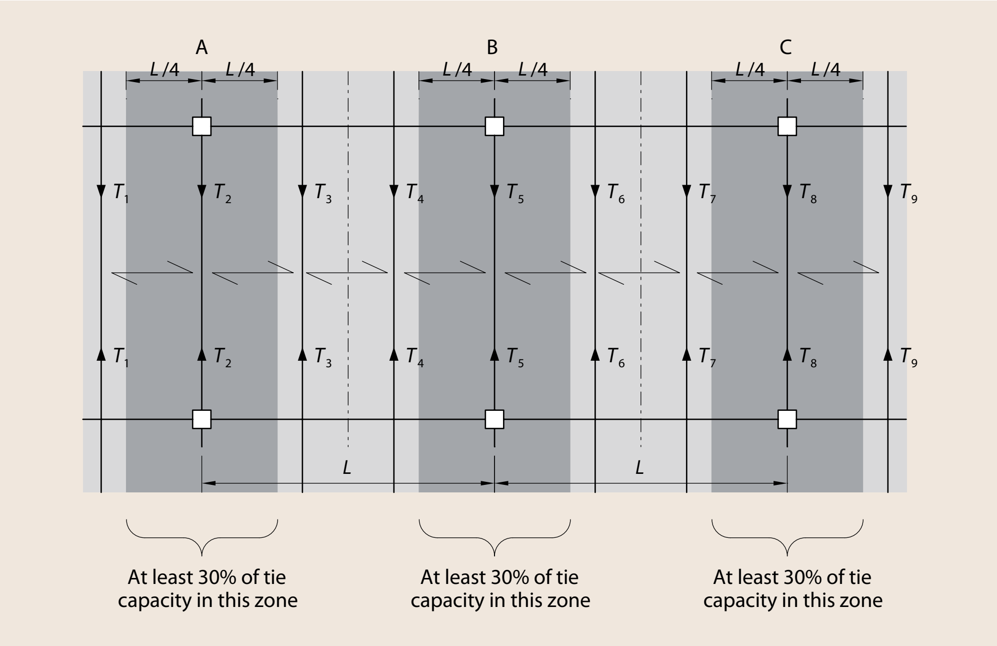


Figure 6.7 Distribution of horizontal tie forces

6.3.6 Tie resistance provided by slab components

As discussed in Section 6.2.2, BS EN 1991-1-7, A.5.1 allows the required tying resistance to be provided by composite slab components such as steel reinforcement or steel sheeting.

It is important to note that the purpose of providing the horizontal ties specified in BS EN 1991-1-7, A.5.1 is to hold the vertical members of the frame in place and to enable the beams to act in catenary. The horizontal tying is not provided to hold the floor slabs in place on the supporting structure, although they will in many cases have this effect. The purpose of the horizontal ties is emphasised by the condition given in BS EN 1991-1-7, A.5.1 (2) that slab components should only be used to provide tying resistance if the composite steel/concrete floors are directly connected to the steel beams with shear connectors (as shown in Figure 6.8). The ties must tie the structural frame together, not just tie floor slabs to adjacent floor slabs. For the arrangement shown in Figure 6.8, the transverse reinforcement, fabric reinforcement and profiled steel decking may all be used to contribute to the required tie force resistance. General guidance on composite slabs and beams is provided in SCI publication P300 [29].

When the tying resistance is provided, entirely or in part, by composite slab components there is scope for the tie force resistance to be distributed across the width of the slab and not located on column lines.

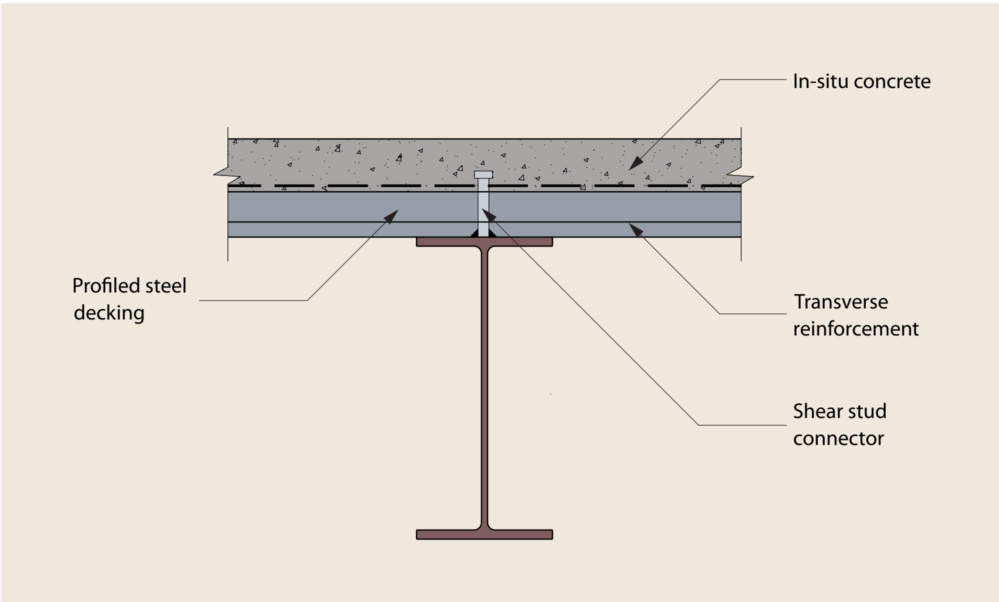


Figure 6.8
Tying with slab components

6.3.7 Beam-to-beam connections

As explained in 6.3.1, BS EN 1991-1-7 and Approved Document A only require a tie member and its end connections to be designed for the required tie force. This requirement can be difficult to interpret when two tie members (beams) connect to the web of a transverse supporting beam from opposite sides, as shown in Figure 6.9. In this situation, the supporting beam web is part of the connection and should be designed to resist the tie forces. Depending on the beam spans and loading, the tie forces on each side of the supporting beam can be of different magnitudes. However, the tie forces are not additive because it is not necessary to consider both tie forces acting simultaneously. Hence, the beam web need only be designed for the larger of the two tie forces, as should the other common components of each connection e.g. the bolts should also be designed for the larger tie force. As discussed in Section 6.3.1, the beam member does not need to be designed to resist lateral bending due to the tie force. The connection shown in Figure 6.9 is an end plate connection but the same principles would apply to fin plate or double angle cleat connections.

SCI publication P358 [25] gives detailed design checks for each component of nominally pinned connections, including those required to calculate the connection tying resistance.

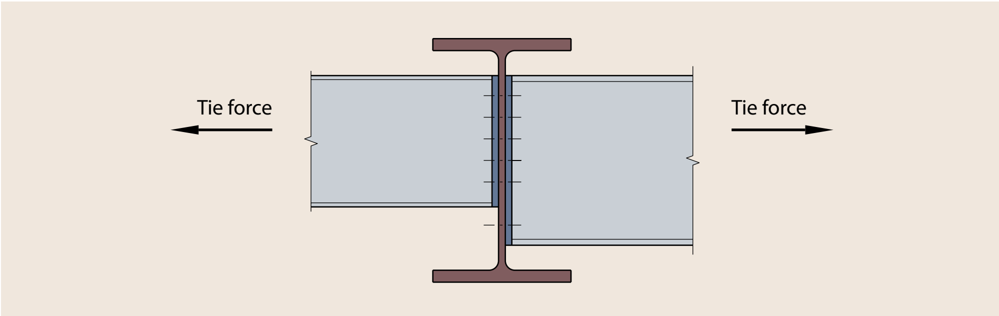


Figure 6.9
Tie force on beam web

6.3.8 Beam arrangements

The tie force equations given in BS EN 1991-1-7, A.5.1 only need to be applied to horizontal members that carry floor loads. Members along with their end connections that do not, in theory, carry any floor load only need to be designed for a tie force of 75 kN. Therefore, different beam arrangements can result in different tying resistance requirements, even though the column grid might be very similar.

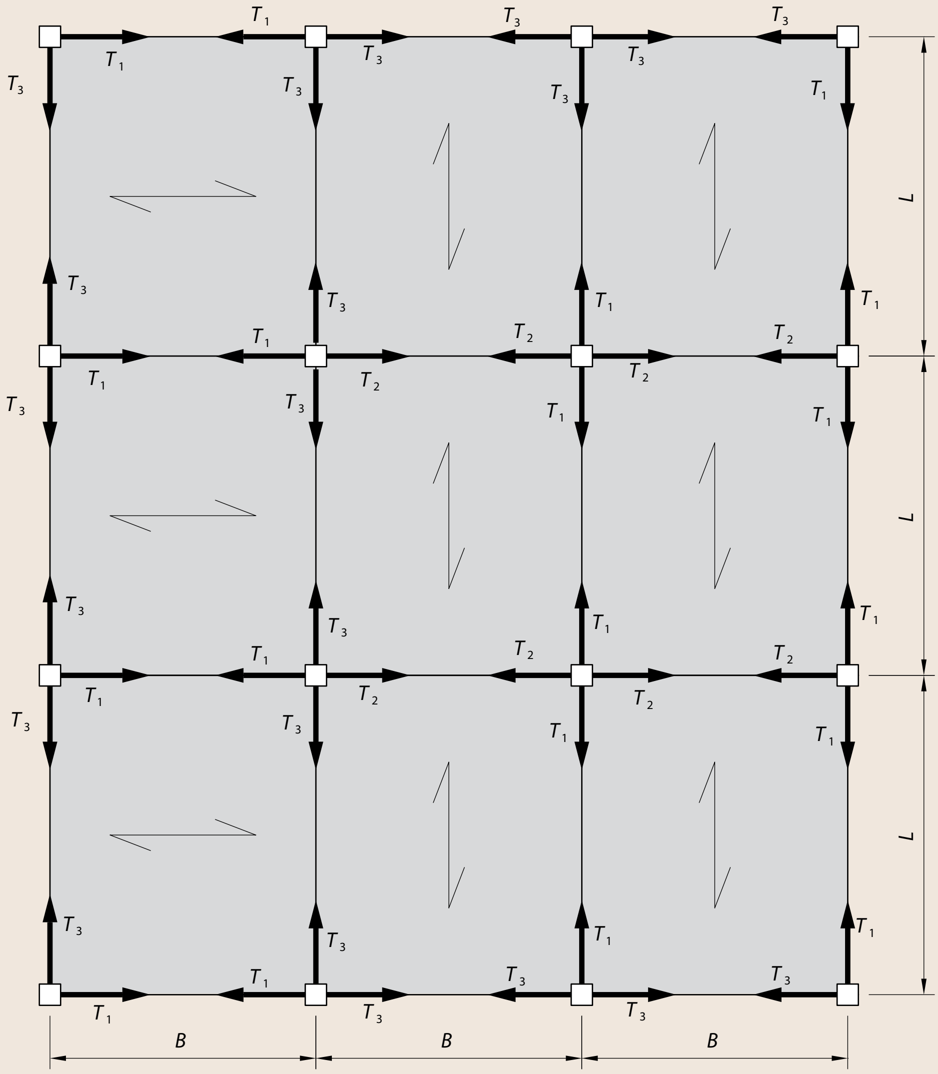
Generic expressions for tying resistance requirements, based on the tie force equations given in BS EN 1991-1-7, A.5.1, for different beam arrangements are shown in Figure 6.10 and Figure 6.11. Figure 6.10 shows tie force expressions for a beam arrangement that could be used with long span slab solutions such as deep composite decking or precast units. Figure 6.11 shows tie force expressions for a beam arrangement that could be used with short span slab solutions such as shallow composite decking.

Table 6.1 gives tying requirements based on the expressions given in Figure 6.10. The maximum tie force calculated for the two scenarios considered in Table 6.1 is 270 kN.

Table 6.1
Tie forces for beam arrangement shown in Figure 6.10

LOADING		TIE FORCE REQUIREMENTS		
g_k kN/m²	q_k kN/m²	T_1 kN	T_2 kN	T_3 kN
3.0	2.5	75	191	96
4.0	4.0	75	270	270

Note: ψ is taken as 0.5, L and B are taken as 7.5 m.

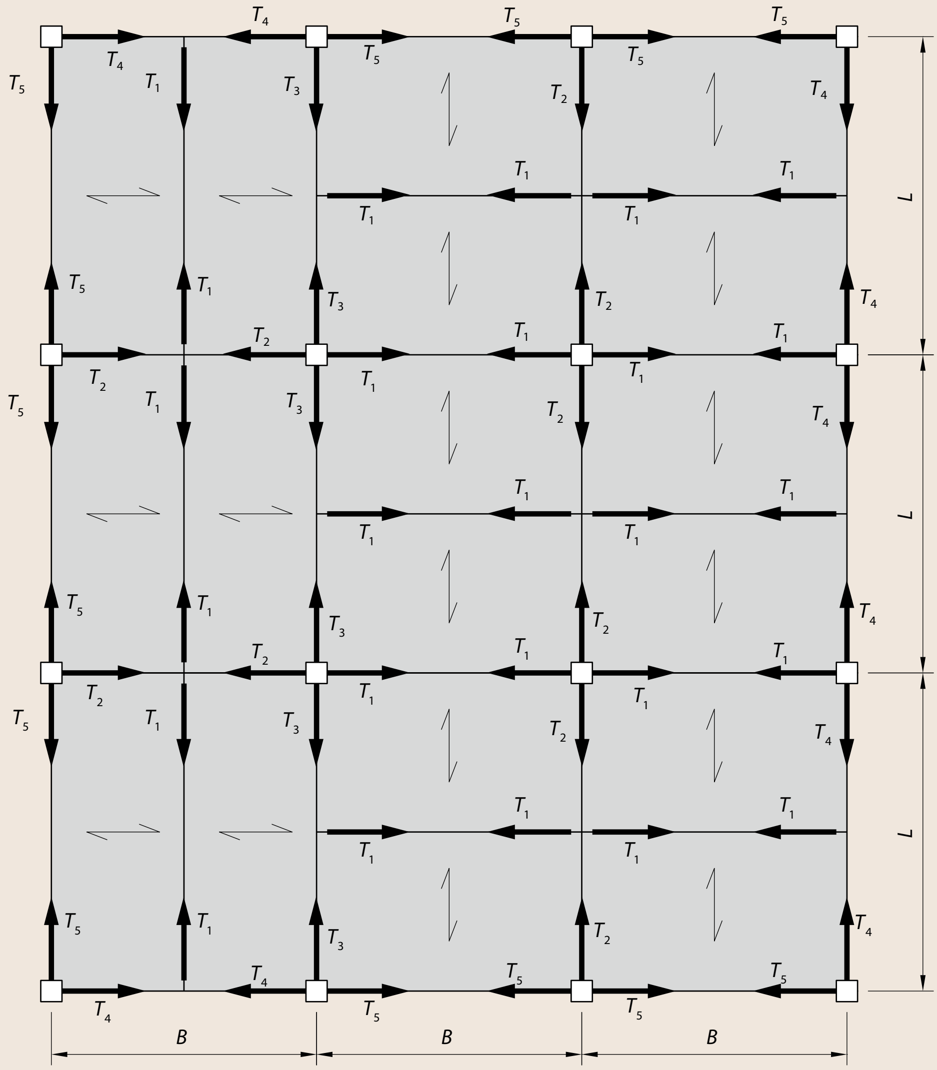


T_1	$= 75\ \text{kN}$	
T_2	$= 0.8(g_k + \psi q_k)LB$	but $\geq 75\ \text{kN}$
T_3	$= 0.4(g_k + \psi q_k)LB$	but $\geq 75\ \text{kN}$

where:

g_k	characteristic dead load
ψ	combination factor for accidental loading
q_k	characteristic imposed load.

Figure 6.10
Generic tie forces for a long span slab beam arrangement



$T_1 \quad = 0.8(g_k + \psi q_k)LB/2$ but ≥ 75 kN

$T_2 \quad = 0.8(g_k + \psi q_k)LB$ but ≥ 75 kN

$T_3 \quad = 0.8(g_k + \psi q_k)3LB/4$ but ≥ 75 kN

$T_4 \quad = 0.4(g_k + \psi q_k)LB$ but ≥ 75 kN

$T_5 \quad = 0.4(g_k + \psi q_k)LB/2$ but ≥ 75 kN

where:

g_k characteristic dead load

ψ combination factor for accidental loading

q_k characteristic imposed load.

Figure 6.11
Generic tie forces for a short span slab beam arrangement

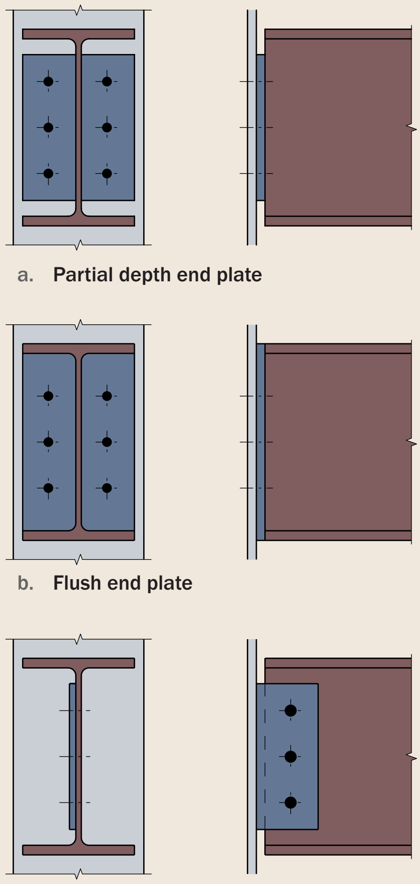
Table 6.2 gives tying requirements based on the expressions given in Figure 6.11. The maximum tie force calculated for the two scenarios considered in Table 6.2 is 270 kN.

Table 6.2
Tie forces for beam arrangement shown in Figure 6.11

LOADING		TIE FORCE REQUIREMENTS				
g_k kN/m²	q_k kN/m²	T_1 kN	T_2 kN	T_3 kN	T_4 kN	T_5 kN
3.0	2.5	96	191	143	96	75
4.0	4.0	135	270	203	135	75

Note: ψ is taken as 0.5, L and B are taken as 7.5 m

Tying resistances for standard connections are given in Figure 6.12; it can be seen that the tie force of 270 kN can easily be satisfied by standard connection details.



Tying resistance with:
3 rows of 2 bolts = 296 to 281 kN
4 rows of 2 bolts = 563 to 374 kN
5 rows of 2 bolts = 699 to 384 kN

a. **Partial depth end plate**

Tying resistance with:
3 rows of 2 bolts = 436 to 421 kN
4 rows of 2 bolts = 741 to 515 kN
5 rows of 2 bolts = 836 to 602 kN

b. **Flush end plate**

Tying resistance with:
3 bolts = 308 to 197 kN
4 bolts = 428 to 289 kN
5 bolts = 535 to 429 kN

c. **Fin plate**

Note: Tying resistances are for standard connection details with S275 beams, taken from SCI-P358 [25]

Figure 6.12
Types of simple connection with typical tying resistances

BUILDINGS IN CONSEQUENCES CLASS 2B

7.1 Structural requirements

7.1.1 Robustness strategy

The recommended strategy in BS EN 1991-1-7, Annex A, A.4 for Consequences Class 2b buildings states:

> *In addition to the recommended strategies for Consequences Class 1, the provision of:*
>
> - *Horizontal ties, as defined in A.5.1 and A.5.2 respectively for framed and load-bearing wall construction (see 1.5.11), together with vertical ties, as defined in A.6, in all supporting columns and walls should be provided.*
>
> *Or, alternatively:*
>
> - *The building should be checked to ensure that upon the notional removal of each supporting column and each beam supporting a column, or any nominal section of load-bearing wall as defined in A.7 (one at a time in each storey of the building) the building remains stable and that any local damage does not exceed a certain limit.*
>
> *Where the notional removal of such columns and sections of walls would result in an extent of damage in excess of the agreed limit, or other such limit specified, then such elements should be designed as a "key element" (see A.8).*
>
> *In the case of buildings of load-bearing wall construction, the notional removal of a section of wall, one at a time, is likely to be the most practical strategy to adopt.*

The scope of this publication is hot-rolled steel frames i.e. framed construction, and not load-bearing wall construction.

Approved Document A

The wording of the Eurocode Recommended Strategy is similar to the guidance given in Approved Document A (2004) for Class 2B buildings. Both documents provide the following three methods for designing Class 2B Buildings for avoidance of disproportionate collapse:

- Tying.
- Notional removal.
- Key element (if notional removal requirements are not satisfied).

It is possible to mix methods within the same building, i.e. a building may generally satisfy the tying method but deal with local areas that do not satisfy the tying method by applying the notional removal or key element methods.

7.1.2 Additional structural provisions

In addition to the robustness strategy from BS EN 1991-1-7, the following good practice is recommended:

- Braced bays or other systems for resisting horizontal forces should be distributed throughout the building such that, in each of two directions approximately at right angles, no substantial portion of the building is connected to only one system for resisting horizontal force.
- Where precast concrete or other heavy floor, stair or roof units are used, they should be effectively anchored in the direction of their span, either to each other over a support or directly to their supports.
- Where the notional removal or key element methods are used, the designer must ensure that the structure is horizontally robust in both directions, which is generally achieved by providing horizontal ties [24].

7.2 Horizontal ties

Where the tying method is used for avoidance of disproportionate collapse, the requirements in BS EN 1991-1-7 for horizontal ties in Class 2b buildings are exactly the same as the requirements for horizontal ties in Class 2a buildings; these are explained in Section 6.2.

7.3 Vertical ties

7.3.1 Benefits of providing vertical ties

Vertical tying resistance is beneficial to a structure in an accidental action situation by allowing loads to be redistributed through the structure via alternative load paths, away from locally damaged areas. This principle is shown in Figure 7.1. Vertical ties also help to limit the risk of the upper floor being blown upwards in an explosion.

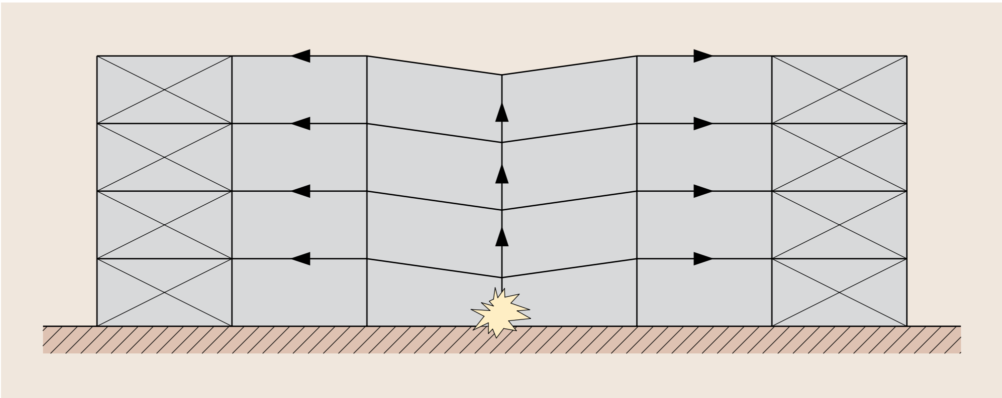


Figure 7.1 Vertical tying allowing loads to find alternative load paths

7.3.2 Design rules

The requirements of vertical ties, as defined in BS EN 1991-1-7, A.6, are given below.

Vertical ties should be:

a. Provided in columns, such that each column is tied continuously from the foundations to the roof level.
b. Capable of resisting a tensile force equal to the largest design vertical permanent and variable load reaction applied to the column from any one storey. Such accidental design loading should not be assumed to act simultaneously with permanent and variable actions that might be acting on the structure.

Vertical tying is provided by the tension resistance of column splices. The vertical tying resistance that is required for column splices is the largest total of the beam end reactions applied to the column at a single floor level. The beam end reactions used to calculate the vertical tying requirement are those from the normal design case (not the accidental load case). For buildings with regular column grids and the same floor loading on each storey, the required column splice tension resistance is equal to the floor area supported by the column multiplied by the design value of the floor loading at ULS.

Guidance on the practical application of the design rules for vertical tying is given in Sections 7.8.1 and 7.8.2.

7.4 Vertical bracing

7.4.1 Benefits of providing vertical bracing

For nominally pin-jointed frames, vertical bracing is required to provide stability to the building and to resist horizontal actions. Vertical bracing can take the form of braced bays or shear walls forming a structural core (e.g. concrete or steel and concrete composite). Vertical bracing is required in two approximately orthogonal directions so that horizontal actions from any direction can be resisted.

The advantage of following the advice in Section 7.1.2 on vertical bracing is relatively clear in that, if one bracing system is damaged by an accidental action, then there will be at least one more bracing system to resist horizontal actions. To reduce the likelihood of all the bracing systems being damaged, the systems can be distributed throughout the structure and not grouped in one location. Good distribution of bracing systems is also beneficial to avoid twisting of the structure on plan when subjected to horizontal actions, in addition to assisting to provide robustness to the structure.

There must be consideration of plan load paths so that horizontal loads have a credible route back to the stability system. The floor system in a steel-framed building is commonly used to transmit horizontal loads to the stability systems. One designer should be responsible for overall stability. If loads have to be chased though floors to get back to cores, it should be clear what that route is and that the floors are capable of diaphragm action. It is equally important that the responsibility for erection is clear, since those diaphragms might not exist during the erection stage.

7.4.2 Design advice

It is advised that vertical bracing to assist in the provision of robustness should comply with the following:

a. Vertical bracing should be distributed throughout the building such that each substantial portion of the building is connected to more than one system for resisting horizontal force in each of two directions approximately at right angles.
b. At least two systems of vertical bracing should be provided in each direction in each substantial part of the building. The combined resistance of the vertical bracing systems should be sufficient to provide lateral stability for the normal design case.

Frames that use moment resisting beam to column connections to provide lateral stability will comply with the above bracing system requirements because, by the nature of the frame, these are distributed throughout the structure. Therefore, the lateral stability of the building is not vulnerable to localised damage.

Guidance on the practical application of the design rules for vertical bracing is given in Sections 7.8.3, 7.8.4 and 7.8.5.

7.5 Anchorage of heavy floor units

7.5.1 Benefits of anchorage of heavy floor units

The intention is to prevent heavy floor units or floor slabs simply falling through the steel frame, if the floor is moved or if the supporting steelwork is moved or removed due to accidental action. Falling floor units or slabs could cause further structural damage and would also cause harm to people in the vicinity. Positive anchorage of floor units prevents them from being displaced due to uplift. The guidance also applies to heavy roof and stair units. It is particularly important to ensure that stair units are suitably anchored to the supporting frame, to ensure they are still in place after an incident, as the stairs will form the means of escape for people inside the building and also the primary route for the emergency services to gain access to the upper floors of the building.

7.5.2 Design advice

It is advised that anchorage of heavy floor, roof and stair units should comply with the following:

a. Heavy floor, roof and stair units should be anchored in the direction of their span.
b. Anchorage should either be directly to the supporting frame or over the supporting frame to an adjacent floor, roof or stair unit.

c. BS EN 1992-1-1 [30] does not cover anchorage of precast floor and roof units and stair members. PD 6687 [31] advises that the same requirements as given in BS 8110 [32] should be used. Precast floor, roof and stair members should be effectively anchored, whether or not such members are used to provide other ties required by BS EN 1992-1-1, 9.10.2.
d. The anchorage should be capable of carrying the self-weight of the member to that part of the structure that contains the ties.

The term 'heavy floor units' was initially used in BS 5950-1 and was interpreted to mean precast concrete units. However, the guidance is applicable to a unit made from any material that could cause significant damage or harm if it were to fall. Hence, there is no definition for what constitutes 'heavy'. Typically, the anchorage requirements are applied to concrete units but designers should also consider that they can be applicable to units made from other materials.

The advice to anchor heavy floor units applies equally to precast concrete floors and to composite beam floors. However, by their nature, composite beam floors with fabric (mesh) reinforcement will generally be adequately anchored.

The safe securing of heavy units during the temporary condition should be considered. Reference 33 provides useful advice on metal anchor fixings.

Guidance on the practical application of the design rules for anchorage of heavy floor and roof units is given in Sections 7.8.6 to 7.8.8.

7.6 Notional removal design strategy

7.6.1 Benefits of notional removal

The notional removal design strategy for robustness in BS EN 1991-1-7, A.4 is presented as an alternative to the provision of horizontal and vertical tying. The benefits of this design strategy are that instead of following prescriptive rules (e.g. tying), more specific damage scenarios are considered, whereby the designer is required to assess the area of the damage, i.e. the building's ability to localise damage. The notional removal design strategy is still somewhat prescriptive in that the damage scenarios that the designer is required to assess involve the removal of one supporting member (beam or column) at a time.

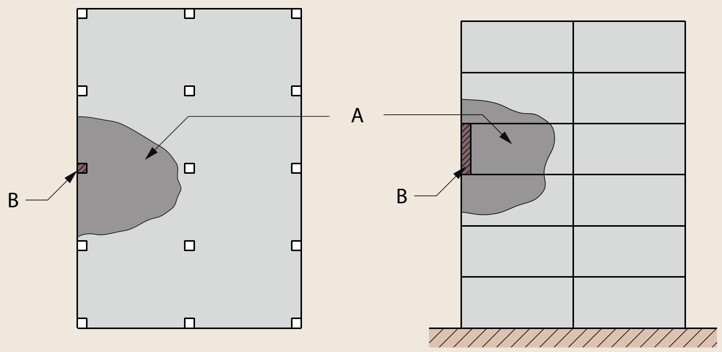
The notional removal design strategy will generally only be successful for small column spacings (see Section 7.6.4).

In practical terms, the advantage is that if the structure has reasonably small beam spans and if the structure is well interconnected, then notional removal offers the designer an opportunity for satisfying robustness rules with an acceptance of local damage. This can be useful if for some reason it is not possible to comply fully with tying rules.

7.6.2 Design strategy

The requirements of the notional removal design strategy as defined in A.4 of BS EN 1991-1-7 are given below.

a. Each supporting member should be notionally removed one at a time to ensure that the limit of admissible local damage is not exceeded and that the building remains stable.
b. For notional removal a supporting member is a column section (a length between adjacent storeys) or a beam supporting one or more columns.
c. The limit of admissible local damage recommended in BS EN 1991-1-7, Annex A, is shown in Figure 7.2. The recommendation is adopted by the UK National Annex. Approved Document A sets a slightly lower limit but is likely to be brought in line with BS EN 1991-1-7 at its next amendment; until that point the lower limit from Approved Document A is applicable for compliance with the UK Building Regulations.
d. Upon the notional removal of any single member, the structure must remain stable as a whole.
e. If the notional removal of any supporting element would result in the collapse of an area greater than the admissible local damage that element should be designed as a key element.
f. If the notional removal of any supporting element would result in the building being unstable that element should be designed as a key element.



a. **Plan** b. **Section**

Figure 7.2 Recommended limit of admissible damage (taken from Figure A.1 of BS EN 1991-1-7

Key
A Local damage not exceeding 15% of the floor area, or 100 m^2, whichever is smaller, in each of two adjacent storeys.
B Notional column to be removed.

Note: The limit of admissible damage in Approved Document A is 70 m^2.

7.6.3 Application of the notional removal strategy

The limit of admissible damage allows for floor area collapse on two separate floors but this should not be taken to mean that floor area collapse will necessarily occur on two floors, or that floor area collapse will not occur on more than two floors. For multi-storey buildings there can be robustness benefits of having designated strong floors positioned periodically at different heights within the building.

Figure 7.2 shows an edge column being notionally removed but the principle of notional removal should also be applied to internal columns and corner columns. Reference 24 suggests that all columns within a plan circular area, with a diameter equal to 2.25 times the storey height, should be notionally removed. The basis for this suggestion is that an accidental event could affect all columns within the local vicinity of the event.

Further guidance on the practical application of the notional removal design strategy is given in Sections 7.6.4 and 7.6.5.

7.6.4 Assessment of damage

The designer is required to determine the degree of structural damage following the notional removal of an element.

To determine the amount of floor area that will collapse on one storey it is normal for a relatively simple approach to be adopted. For the case of a column being notionally removed, all the beams supported by the column are assumed to collapse and all the floor slabs supported by the collapsed beams are assumed to collapse. The smallest column grid that might be expected in a steel frame building is 6 m × 6 m (as shown in Figure 7.3) which would result in a 144 m^2 area of floor collapse and thus exceed the limit of admissible local damage. Therefore, notional removal of internal columns is unlikely to be a viable design strategy for steel frame buildings. However, the strategy could be successful for edge or corner columns where the resulting area of floor collapse is less.

If the total area of floor slab that is assumed to collapse is greater than the admissible local damage (i.e. minimum of 15% or 100 m^2) then the notional removal design strategy cannot be used for this column section.

If the total area of floor slab that is assumed to collapse is less than the admissible local damage then the designer should check that floor area collapse occurs on no more than two floors.

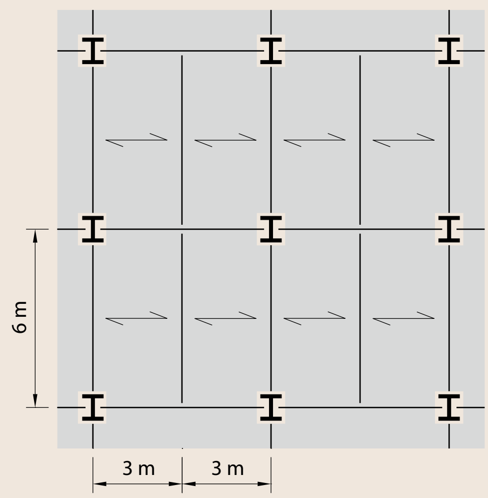


Figure 7.3 Beam arrangement for 6 m x 6 m column grid

To ensure the damage does not spread down the building, the floor beneath the collapsed floor should be checked to ensure that it does not collapse due to the debris load from the floor above falling onto it. The load case for this check should be the same as the load case used for all the notional removal checks (see Section 7.6.5). There is no need to include a dynamic amplification factor for the debris loading because this is a notional design check and the load can be taken as evenly distributed (i.e. not heaped).

To ensure the damage does not spread up the building, the floors above the notionally removed column should be checked to determine whether they can bridge over the removed column. Without horizontal and vertical tying and sufficient systems capable of resisting lateral forces the floors above will also collapse.

The stages of the notional removal design approach are shown diagrammatically in Figure 7.4.

When the notional removal of a supporting member results in damage exceeding the admissible limit, the designer has three options:

a. Design the supporting element as a key element (see Section 7.7).
b. Modify the structure so that the amount of resulting damage is reduced below the admissible limit.
c. Adopt the tying method through the provision of horizontal tying, vertical tying, vertical bracing and anchorage of heavy floor, roof and stair units.

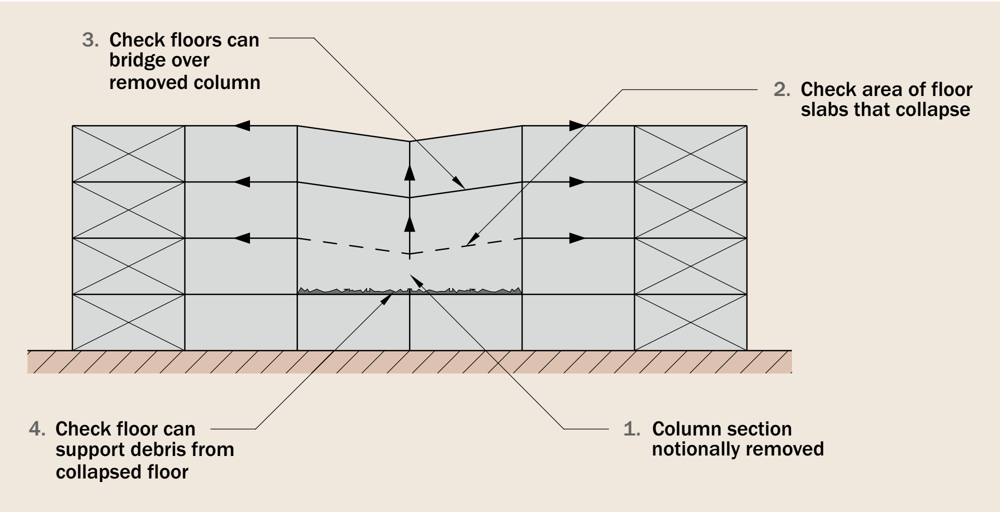


Figure 7.4 Notional removal design approach

7.6.5 Combination of actions for notional removal

The combination of actions for accidental design situations is given in expression BS EN 1990 6.4.3.2, 6.11b as:

$$\sum_{j\geq 1} G_{k,j} + P + A_d + (\psi_{1,1}\,\mathrm{or}\,\psi_{2,1})Q_{k,1} + \sum_{i>1}\psi_{2,i}Q_{k,i}$$

This combination includes the same actions as for the fundamental combination and also the design value of the accidental action A_d. For the situation after an accidental event $A_d = 0$. The partial factors on the other actions are all equal to unity and are therefore not shown. All variable actions are taken to be accompanying actions and the factors for frequent values (ψ_1) or quasi-permanent values (ψ_2) are applied. For the option of $\psi_{1,1}$ or $\psi_{2,1}$, the UK National Annex [34] says that ψ_1 should be used. Values for ψ_1 and ψ_2 are given in the National Annex.

The combination of actions for the notional removal design approach check is expressed as:

$$\sum_{j\geq 1} G_{k,j} + \psi_{1,1}\, Q_{k,1} + \sum_{i>1} \psi_{2,i} Q_{k,i}$$

where:

$G_{k,j}$	are the characteristic values of the permanent actions
$Q_{k,1}$	is the characteristic value of one of the variable actions
$Q_{k,i}$	are the characteristic values of the other variable actions
$\psi_{1,1}$	is the factor for the frequent value of the variable action $Q_{k,i}$ (see Table 3.1)
$\psi_{2,i}$	is the factor for the quasi-permanent value of the variable action $Q_{k,i}$ (see Table 3.1).

7.7 Key element design

7.7.1 Benefits of key element design

The key element design approach may be applied where the notional removal of a supporting member would result in the limit of admissible damage being exceeded.

The key element approach is fundamentally different from the tying approach and the notional removal approach. Both the tying approach and the notional removal approach are focused on limiting the spread of damage, or collapse, following an event that has caused a supporting element to be damaged. In contrast, the key element approach is focused on preventing the supporting element being damaged (to an extent that it can not provide the required support) following an accidental event and thus preventing excessive failure.

7.7.2 Design rules

The requirements of key element design as defined in A.8 of BS EN 1991-1-7 are given below:

a. Key elements should be capable of sustaining an accidental design action of A_d applied in horizontal and vertical directions (in one direction at a time) to the member and any attached components.
b. The recommended value of A_d for building structures is 34 kN/m^2.
c. The accidental design action should be applied to the key element and any attached components having regard for the ultimate strength of attached components and their connections.

d. The accidental design loading should be applied in accordance with expression (6.11b) of EN 1990.

Guidance on the practical application of the design rules for key element design is given in Sections 7.8.10, 7.8.11, 7.8.12 and 7.8.13.

7.8 Practical application of design rules

7.8.1 Continuous vertical tying

The requirements in A.6 refer to tying columns from foundations to roof level. In the majority of ordinary buildings, columns will be present from foundations to roof level. However, in some buildings, not all the columns will run from foundations to roof level due to the use of transfer beams (i.e. beams supporting one or more columns), cantilever beams and changes in column grids; some examples are shown in the frame elevation in Figure 7.5. Guidance for providing robustness in frames with transfer structures is given in Section 9.

Columns in Class 2b buildings should be tied vertically from their base to their top whether or not these coincide with the foundation and roof level. Where the base of a column is not at the foundation level, the base of the column should be tied vertically to the structural frame at that level. Where the top of a column is not at the roof level, the top of the column should be tied vertically to the structural frame at that level.

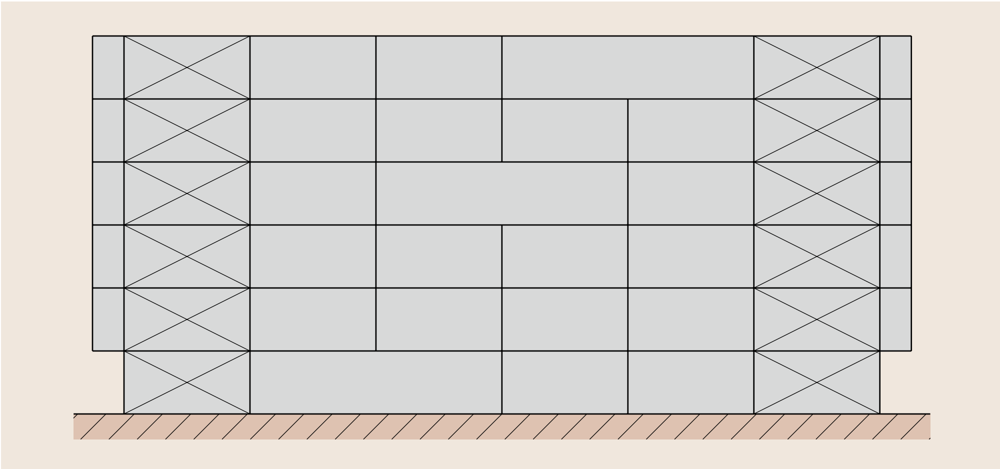


Figure 7.5 Columns not from foundations to roof level

7.8.2 Column splice tying resistance

In practice, providing vertical tying is unlikely to be an onerous obligation, as most splices designed for adequate stiffness and robustness during erection are likely to be sufficient to carry the axial tying force. SCI publication P358 [25] gives details of standard splices and quotes axial tension resistances to simplify the design checks. Either bearing or non-bearing column splices (as shown in Figure 7.6) can be designed to satisfy the vertical tying requirements. Non-bearing splices will generally have higher tension resistance because they require thicker cover plates and more bolts for normal design.

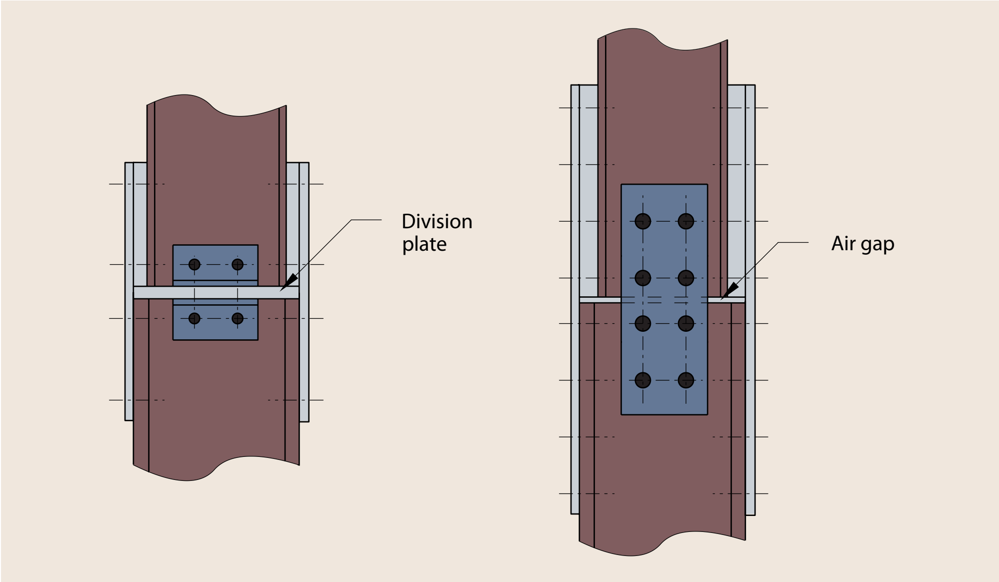


Figure 7.6
Column splice details

Table 7.1 gives indicative tensile axial resistances for standard bearing-type column splices with cover plates.

The resistances quoted in Table 7.1 are limited by bolt shear; adding additional bolts will increase the resistances. Detailed design checks for bearing and non-bearing column splices are provided in Reference 25.

It is likely to be more difficult to provide the necessary tensile resistance with ‘cap and base’ type column splices, such as shown Figure 7.7. Design of such connections should ensure that the vertical tying requirement is satisfied. If necessary, the connection type should be changed to provide greater tensile resistance.

Table 7.1
Typical bearing type column splice tensile resistances (with flange cover plates)

UPPER COLUMN	LOWER COLUMN	TENSILE RESISTANCE
152 × 152 UC	152 × 152 UC	≥ 244 kN
203 × 203 UC	203 × 203 UC	≥ 312 kN
254 × 254 UC	254 × 254 UC	≥ 319 kN
305 × 305 UC	305 × 305 UC	≥ 632 kN
356 × 368 UC	356 × 368 UC	≥ 797 kN
356 × 406 UC	356 × 406 UC	≥ 529 kN

Source: Reference 25

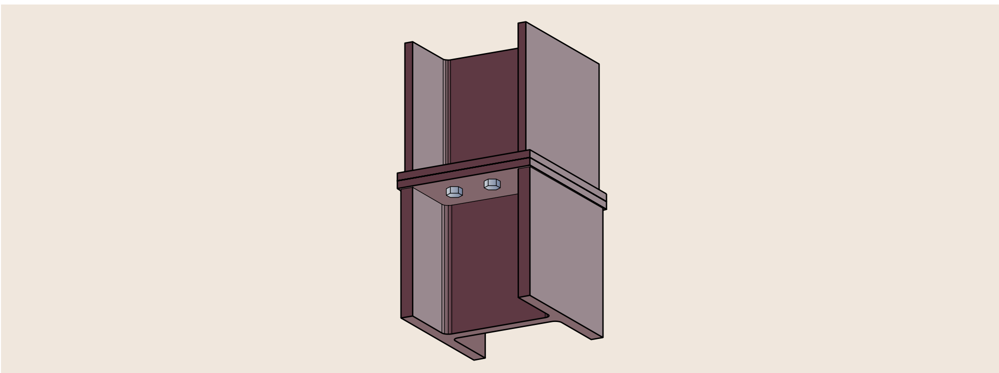


Figure 7.7
Cap and base column splice

7.8.3 Design of bracing systems

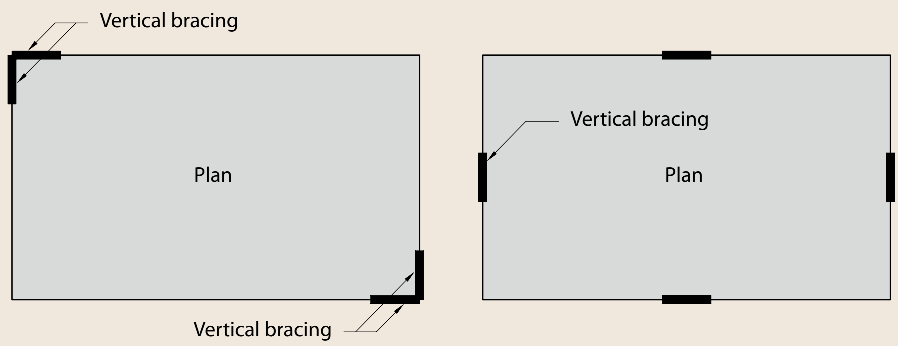
The advice on bracing systems, given in Section 7.4.2, relates to the number and distribution of systems, not to their design resistance. In other words, the advice given does not suggest that additional design resistance should be provided; just that the required bracing resistance should be suitably distributed.

As stated in Section 7.4.1, the reason for distributing the bracing systems throughout the structure is to reduce the likelihood of more than one system being damaged by one accidental event. Therefore, in the post-accidental event situation, there are other bracing systems that could provide lateral stability. Generally all the bracing systems will be needed to provide sufficient lateral stability to the structure for the 'normal' design case. However, in the accidental design case the structure can have sufficient lateral stability, even if one of the bracing systems is damaged, because the load factors are lower in the accidental design situation.

It is not intended, or advised, that designers should be considering scenarios where one bracing system is damaged and the combined resistance of the remaining bracing systems is checked for adequacy against an accidental design case.

7.8.4 Location of bracing systems

Figure 7.8 shows the plan view of two buildings; each building satisfies the advice for having at least two sets of vertical bracing in each orthogonal direction. The bracing arrangement shown in Figure 7.8 b is less vulnerable to damage due to an accidental action than the arrangement shown in Figure 7.8 a because the bracings are not located close to the corners. One accidental action near to one of the corners with vertical bracing in Figure 7.8 a could cause damage to two sets of vertical bracing, whereas it would require a much larger accidental action to cause damage to two sets of vertical bracing in Figure 7.8 b. Hence, the arrangement shown in Figure 7.8 b will generally provide a more robust structure than the arrangement shown in Figure 7.8 a. In practice, the exact location of possible accidental actions (e.g. the movement of vehicle close to, or inside, the building) will be a factor in determining the least vulnerable location for bracing systems.



a. **Vulnerable location for bracing near to corners**

b. **Less vulnerable location for bracing remote from corners**

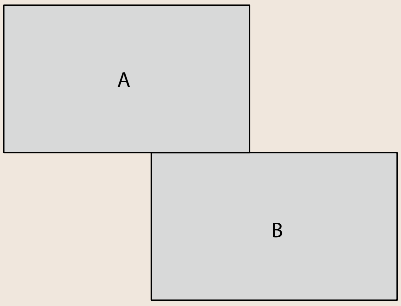
Figure 7.8 Location of vertical bracing

Note that, in the advice on bracing systems in Section 7.1.2, it says that "*no substantial portion of the building is connected to only one system for resisting horizontal force*". A definition for what constitutes a substantial portion is deliberately omitted because this will be different for each building. Some examples and suggestions of what could be a substantial portion are discussed below and shown in Figure 7.9.

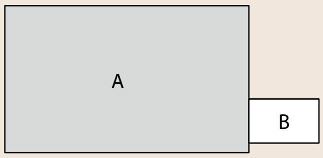
The building plan shown in Figure 7.9 a can be divided into two rectangular parts each representing a substantial portion of the building and therefore each should have at least two sets of vertical bracing in each orthogonal direction. If one of the portions was drastically smaller in size than the other, as shown in Figure 7.9 b, then it could be considered that the smaller part was a non substantial portion and therefore it would not need to have at least two sets of vertical bracing in each orthogonal direction.

The building plan shown in Figure 7.9 c can be divided into three rectangular parts. The two larger parts, A and C, each represent a substantial portion of the building and therefore each should have at least two sets of vertical bracing in each orthogonal direction. The smaller part, B, could be claimed to be a non substantial part and therefore would not need to have at least two sets of vertical bracing in each orthogonal direction. Whether Part B would be a substantial portion would depend on its relative size to the other two parts.

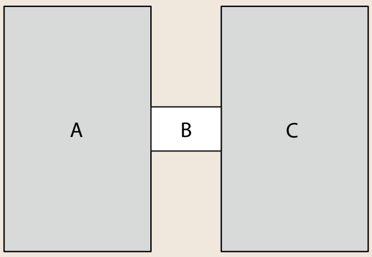
The building plan shown in Figure 7.9 d can be divided into three rectangular parts, each representing a substantial portion of the building and therefore each should have at least two sets of vertical bracing in each orthogonal direction.



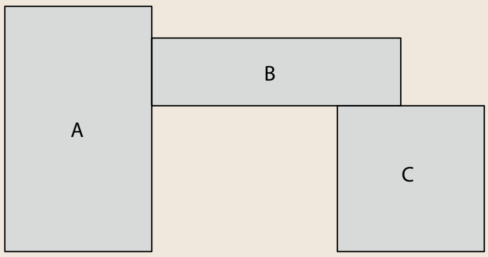
a. **Building with two substantial portions**



b. **Building with one substantial portion and one non-substantial portion**



c. **Building with two substantial portions and one non-substantial portion**



d. **Building with three substantial portions**

Note: Shaded areas show substantial portions, non-shaded areas show non-substantial portions.

Figure 7.9 Examples of building portions

When considering the number of bracing systems required for robustness, the plan dimensions of the building can be reasonably considered as a factor. For a building with small plan dimensions it could be reasonable not to provide two sets of vertical bracing in each direction. However, it is important to consider each case on its merits.

7.8.5 Concrete cores

The functions of vertical bracing systems can be provided partially or entirely by one or more reinforced concrete cores or by *Corefast* [35] cores. A concrete core normally consists of four walls; each wall has the capability to provide lateral stability in the direction of the plane of the wall. It is usual for there to be openings in the walls of concrete cores (e.g. to provide access to lifts). Openings will reduce the ability of the wall to resist lateral stability. It is unlikely that the entire wall will be omitted for an opening; the remaining parts of the wall will provide some resistance to lateral loads. Depending on the detailing of the concrete core and the number and size of openings, it may or may not constitute two bracing systems in each of the two orthogonal directions. Hence, in order to satisfy the recommendation to provide at least two sets of vertical bracing in each orthogonal direction it might be necessary to provide more than one concrete core.

7.8.6 Anchorage of precast units

Anchorage across internal supports

To anchor precast units over internal supporting beams it is possible to expose the voids in the precast planks and place reinforcing bars between the two units prior to concreting, as shown in Figure 7.10.

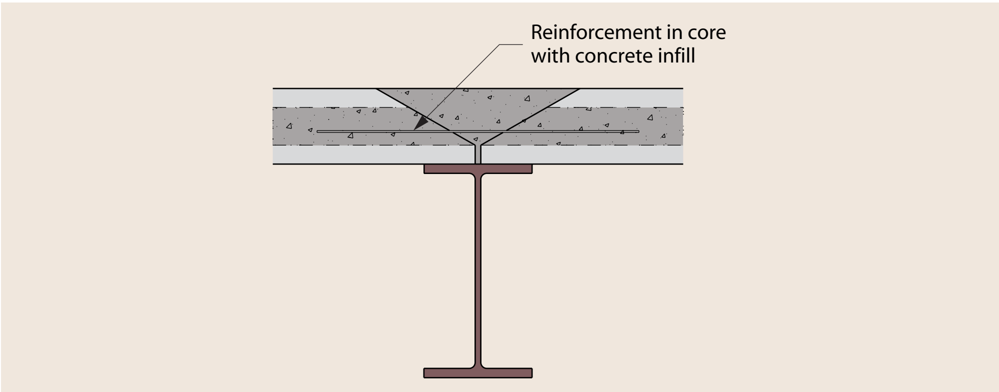


Figure 7.10 Anchorage of precast units over supporting beams

Special measures will be needed where precast planks are placed on shelf angles, as shown in Figure 7.11, and with asymmetric beams (see Figure 7.12), unless the anchorage forces can be carried through the reinforcement in the screed, assuming this is above the top flange of the steelwork. When it is not possible to use reinforcement in the screed, straight reinforcement bars anchoring the precast units together are usually detailed to pass through holes drilled in the steel beam. The practical placement of bars through holes in the beam web needs careful consideration.

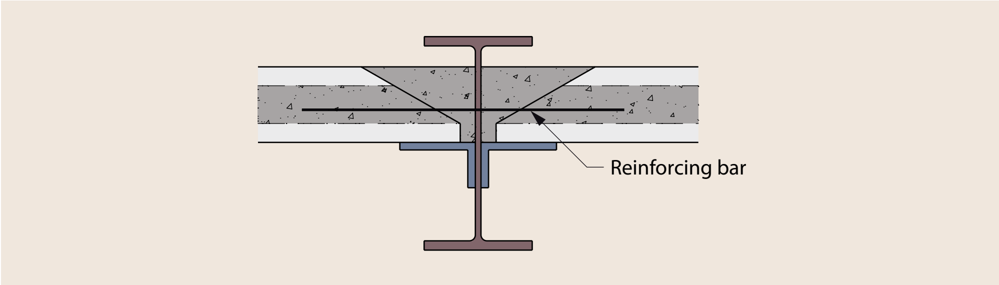


Figure 7.11
Anchorage of precast units on shelf angles

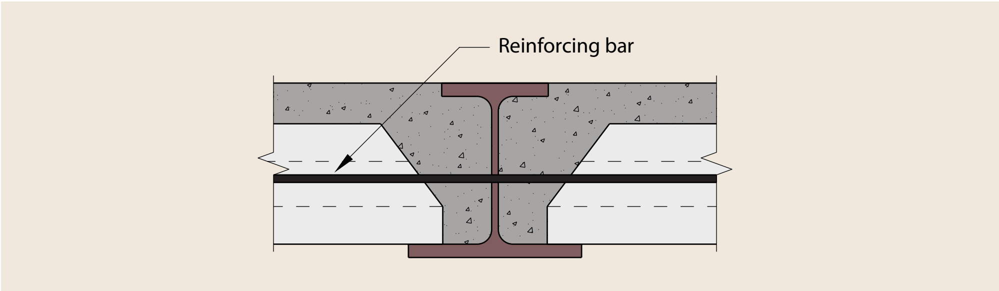


Figure 7.12
Anchorage of precast units on ASB

Anchorage to edge beams

Anchorage is best accomplished by exposing the voids in the plank, and placing U-shaped bars around studs welded to the steelwork, as shown in Figure 7.13. In this Figure, the studs have been provided in order to achieve adequate anchorage, not for composite design of the edge beam. Figure 7.13 b is a more complicated solution involving castellation of the plank edge, (often on site) so that the plank fits around the stud, and similar U-bars located in the voids prior to concreting. The minimum widths shown in Figure 7.13 are typical but the actual dimension depends on the type of plank (solid or hollow core), the end detail of the plank (square end or chamfered), the span of the plank and whether the studs on the beam have been shop or site welded. Guidance on the minimum dimensions for the varying situations is given in Reference 36.

It should be noted that loading a beam only on one side produces significant torsion in the beam itself, which might well be the critical design case. The eccentricity must be accounted for in the design of the member, connections and columns.

Special consideration may need to be given to floor units that cantilever past the edge beam.

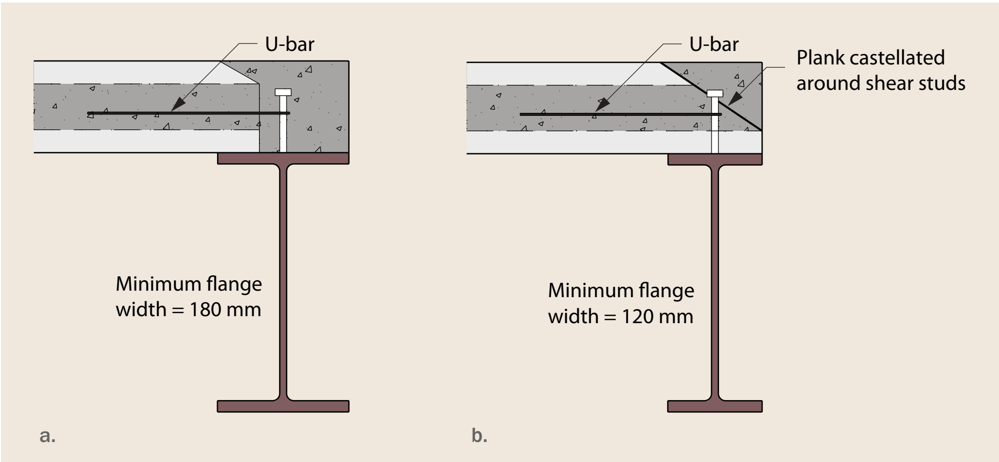


Figure 7.13
Anchorage of precast planks to edge beams

7.8.7 Anchorage of composite slabs with metal decking

Anchorage across internal supports

Internal beams supporting composite slabs are generally designed as composite beams. In such cases, the anchorage requirement will be satisfied by the form of construction, without the need for additional measures to ensure anchorage.

Anchorage to edge beams

Where edge beams are designed as composite beams the floor slab anchorage requirements will generally be satisfied without the need for additional reinforcement or shear studs.

Where edge beams of composite slabs are designed as non-composite beams, the floor slab must still be anchored to the supporting edge beam. There are several methods that can be used to anchor the slab to the beam. The placement of U-shaped bars around studs welded to the edge beam may be used to anchor the slab. If through deck welding has been used to fix the studs to the edge beam, the resistance of the connection between the deck and beam alone may be sufficient for anchorage without the need for U-bars. The anchorage provided in each case should be quantified, so that its adequacy can be verified.

7.8.8 Anchorage of *Slimdek* floor slabs

The *Slimdek* floor system comprises composite floor slabs formed on deep metal decking that is supported on the bottom flange of asymmetric beam sections (see Figure 7.14). Further details of the floor system are provided in Reference 37 and SCI publication P392 [38].

The construction of the *Slimdek* system and in particular the integration of the floor beams into the depth of the composite slab means that there are advantages in terms of robustness. SCI report RT1215 [39] examines the *Slimdek* floor system and presents modified robustness rules. As that report relates to structural design using BS 5950, the guidance presented below has been adapted from that given in the report so as to suit Eurocode design requirements.

Anchorage across internal supports

The reinforcement provided to anchor slabs together over supports must be capable of supporting the weight of the slab in the event of a collapse but it can be the same reinforcement as that used to prevent cracking in the slabs, provided the reinforcement is continuous over or through the beam or tie member. A142 fabric is usually provided in *Slimdek* slabs as a minimum, which is adequate for the majority of situations.

Anchorage to edge beams

The anchorage required at edge beams depends on the anchorage that is provided to the slab on the other edges:

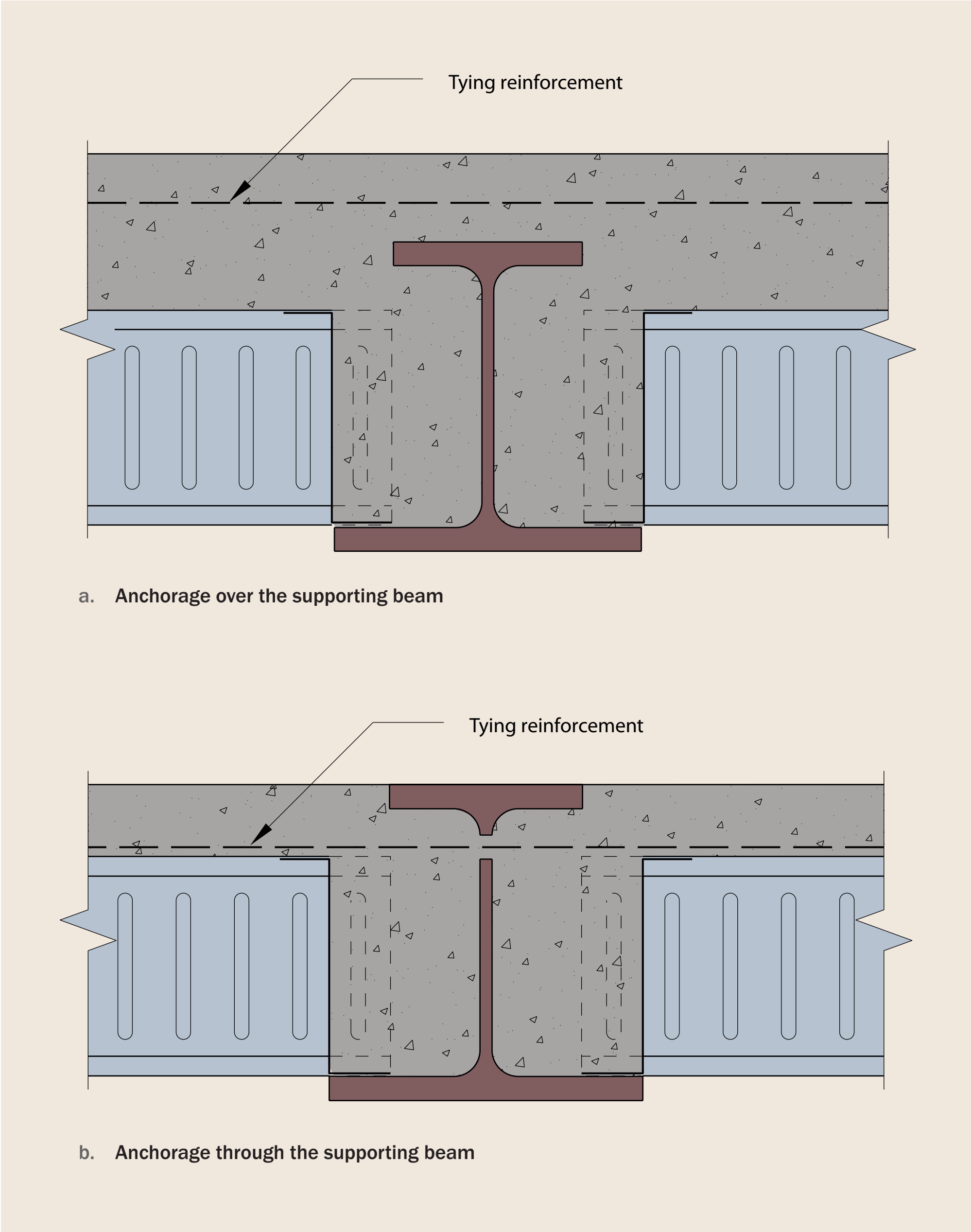


Figure 7.14
Slimdek floor system

- Slabs spanning onto an edge beam (but not corner slabs) need not be anchored to the edge beam provided that anchorage is provided on the other three sides of the slab.
- Corner slabs need not be anchored to the edge beam provided that anchorage is provided along the two internal edges of the slab.

Anchorage required at edge beams can be achieved in different ways, depending on the edge beam section type. Figure 7.15 shows three possible solutions for edge beam anchorage:

- A composite RHS edge beam with anchorage provided by ‘U’ bars around shear connectors.
- A composite ASB edge beam with anchorage provided by ‘L’ bars over the ASB.
- A downstand edge beam with anchorage provided by ‘U’ bars around shear connectors.

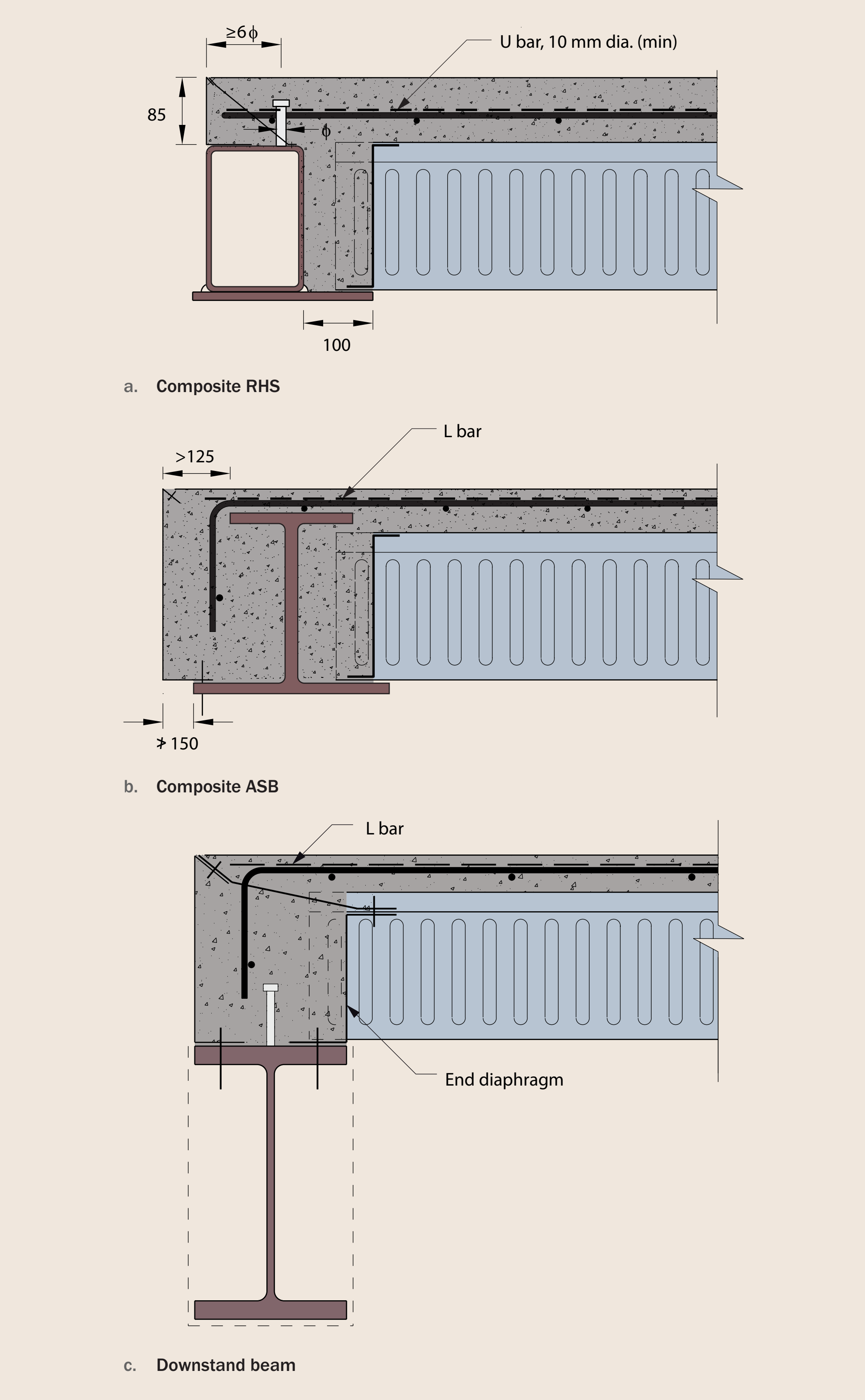
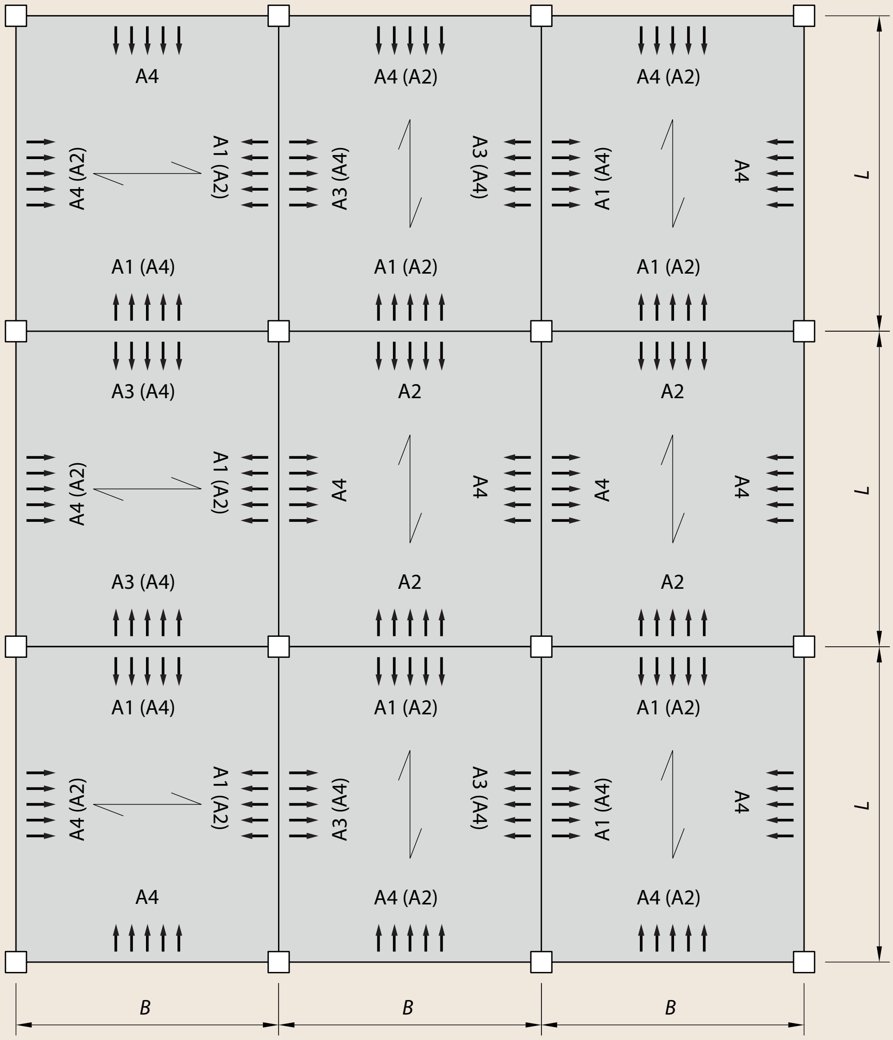


Figure 7.15 Typical anchorage provision in edge beams with Slimdek

7.8.9 Slab anchorage requirements

Slab anchorage requirements are summarised in Figure 7.16. The Figure is applicable to floor slabs formed from precast units or concrete and metal decking.



$A1$	$= gLB$	
$A2$	$= gLB/2$	
$A3$	$= gLB/3$	
$A4$	$= 0$	(no anchorage required)

where:

g characteristic self-weight of floor slab / m^2

Notes: Where there are two alternative anchorage values presented, the selection must be consistent within each slab (i.e., if the first option is selected for one side of the slab the first option must be selected for all other sides of the slab).

Where the slab anchorage requirements are different on either side of the interface between two slabs, the anchorage provided across the interface must be capable of resisting the larger of the two anchorage forces.

Figure 7.16 Slab anchorage requirements

7.8.10 Key element accidental design action

The recommended value of the accidental design action (A_d) for key elements is given in BS EN 1991-1-7, A.8 as 34 kN/m^2.

The accidental design action is intended to represent a range of possible accidental events including impacts and explosions and is used as a tool for designing key elements to be more robust than is required for normal design cases. The origin of the

34 kN/m^2 relates to the Ronan Point collapse in London, 1968, where a domestic gas explosion caused the disproportionate collapse of a 23 storey precast concrete block of flats. Post collapse analysis of the structure estimated that the maximum static equivalent pressure from the explosion was 34 kN/m^2.

For some key elements it could be appropriate to consider other accidental actions that might occur e.g. vehicle impact for perimeter columns. Guidance on vehicle impact forces is given in BS EN 1991-1-7, 4.3.

7.8.11 Components attached to key elements

BS EN 1997-1-7, A.8 clearly states that the accidental design action (A_d) should be applied to the key element and any attached components having regard for the ultimate strength of attached components and their connections. Therefore, the 34 kN/m^2 should be applied to the key element and any components attached to the key element, unless the attached components or their connections cannot sustain the 34 kN/m^2. Hence, for the design of a key element, it is necessary to consider what components, or proportion of components, will remain attached to the element in the event of an incident. The application of engineering judgement will play a major part in this process.

For framed construction, the walls and cladding will normally be non-structural. Therefore, it is likely that the majority of these will become detached from the key element during an incident, as shown in Figure 7.17. For the column member key element shown in Figure 7.17, an accidental load of 34 kN/m^2 should be applied over a width b_{eff} for accidental loading about the major axis. The column section should be checked for the combination of moments and axial force using the design case given in Section 7.8.13. The accidental loading about the minor axis over a width of h (in this case) also needs to be considered. The accidental loading should only be considered as acting in one direction at a time and there is no requirement to consider a diagonal loading case i.e. at an angle to the major and minor axes. However, in reality, an accidental action could occur in any direction and potentially in more than one direction at a time depending on the cause of the action.

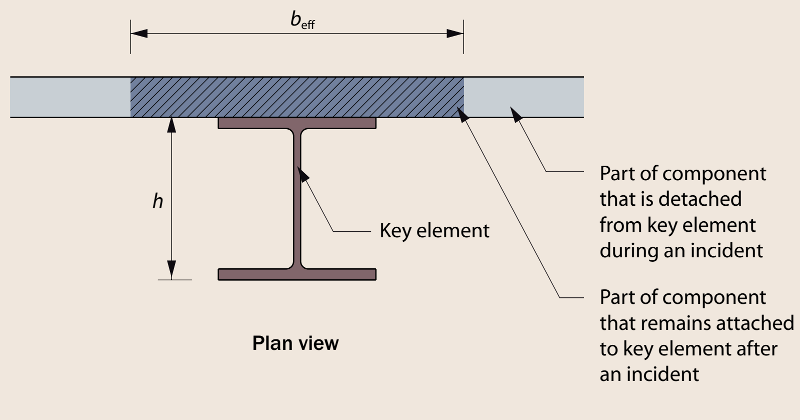


Figure 7.17 Component attached to a key element (column)

Determining the width b_{eff} is very subjective. An estimation of what will remain attached to the key element (during a load of 34 kN/m^2) will obviously depend on what is attached and how it is fixed to the element.

Any components normally attached to the key element but which cannot sustain the 34 kN/m^2 cannot be used to restrain the key element in any way for the accidental design case. Therefore, any structural component that provides restraint vital to the stability of a key element for the accidental design case should also be designed as a key element. However, the key element is unlikely to require the same level of restraint for the accidental design case as it does for the normal design case because the factors on the permanent and variable actions are lower.

Where planar elements connected to a key element would become detached from the key element at a load less than 34 kN/m^2, a lower accidental design action should also be considered by the designer as this could represent a more severe design case for the key element. This is demonstrated by the key element column section shown in Figure 7.18. The key element shown in Figure 7.18 is a 3 m high column, which is connected to a wall with columns spaced a 6 m centres. For the case where the accidental design action is taken as the recommended value of 34 kN/m^2 it is assumed that the wall cannot resist the accidental action and a nominal 1 m wide strip is assumed to remain connected to the key element. Therefore, the total accidental force on the key element is $34 \text{ kN/m}^2 \times 1 \text{ m} \times 3 \text{ m} = 102 \text{ kN}$. However, for a lower accidental design action of 10 kN/m^2 it could be assumed that the wall can resist the accidental action and the complete wall remains connected to the key element. Then, the total accidental force on the key element is $10 \text{ kN/m}^2 \times 6 \text{ m} \times 3 \text{ m} = 180 \text{ kN}$, which is a more severe design case for the key element.

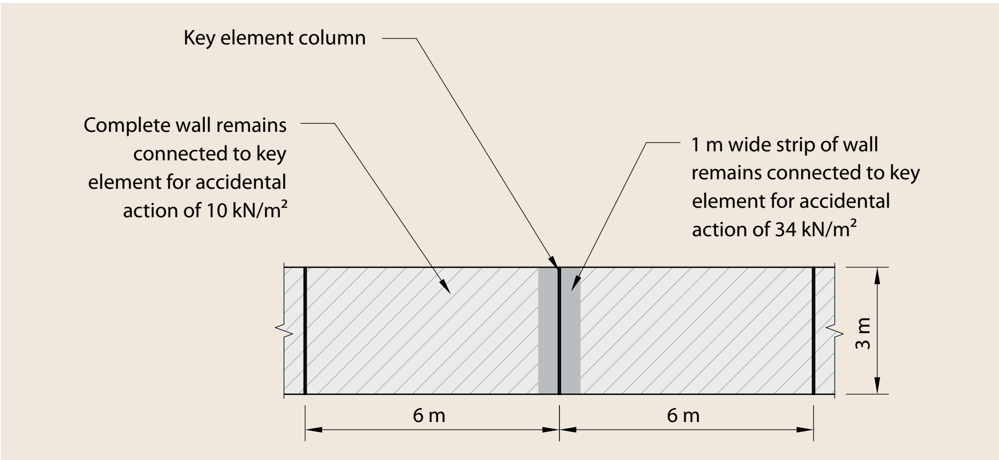


Figure 7.18 Wall connected to key element column

7.8.12 Accidental loading on large areas

When considering the accidental loading on a large area (e.g. on a floor slab supported by a transfer beam), it is reasonable to limit the area that is subjected to the 34 kN/m^2 load because a blast pressure is unlikely to be this high on all the surfaces of a large enclosed space. The maximum area is not defined but could be inferred from the length of load-bearing wall to be considered (see BS EN 1991-1-7, A.7), which is 2.25 times the storey height, say $2.25 \times 2.9 = 6.5$ m. Therefore, a maximum area that would be subjected to the 34 kN/m^2 could be a 6.5 × 6.5 m square.

Note: Reference 24 suggests a maximum area of 6 m × 6 m, but also makes the point that each case should be considered in light of the specific circumstances.

7.8.13 Combination of actions for key element design

Key elements and attached components must be designed for an accidental design case that is the same as the design case for the notional element design strategy except that it includes an accidental action of 34 kN/m^2.

The combination of actions for the design of key elements and attached components is expressed as:

$$\sum_{j \geq 1} G_{k,j} + A_d + \psi_{1,1} Q_{k,1} + \sum_{i > 1} \psi_{2,i} Q_{k,i}$$

where:

$G_{k,j}$	are the characteristic values of the permanent actions
A_d	is the accidental action, based on the recommended value of 34 kN/m^2
$Q_{k,1}$	is the characteristic value of one of the variable actions
$Q_{k,i}$	are the characteristic values of the other variable actions
$\psi_{1,1}$	is the factor for the frequent value of the variable action $Q_{k,i}$ (see Table 3.1)
$\psi_{2,i}$	is the factor for the quasi-permanent value of the variable action $Q_{k,i}$ (see Table 3.1). Note: ψ_2 is zero for all actions likely to be accompanying.

The combination of actions for the remaining parts of the structure that are not subjected to the accidental action of 34 kN/m^2 is as for the notional removal design strategy (see Section 7.6.5).

BUILDINGS IN CONSEQUENCES CLASS 3

8.1 Structural requirements

8.1.1 Robustness strategy

The recommended strategy in BS EN 1991-1-7, Annex A, A.4 for Consequences Class 3 buildings states:

> *A systematic risk assessment of the building should be undertaken taking into account both foreseeable and unforeseeable hazards.*
>
> *Guidance on risk analysis is included in Annex B.*

Annex B of BS EN 1991-1-7 provides an informative annex that offers guidance for the planning and execution of risk assessments. The UK National Annex to BS EN 1991-1-7 declares that Annex B may be used where alternative provisions are not included in the body of BS EN 1991-1-7 and therefore, Annex B is the recommended strategy for Class 3 buildings in the UK.

Approved Document A

The wording of Annex A, A.4 is similar to the guidance given in Approved Document A (2004) for Class 3 buildings except that Approved Document A goes a little further by saying:

> *For Class 3 buildings - A systematic risk assessment of the building should be undertaken taking into account all the normal hazards that may reasonably be foreseen, together with any abnormal hazards.*
>
> *Critical situations for design should be selected that reflect the conditions that can reasonably be foreseen as possible during the life of the building. The structural form and concept and any protective measures should then be chosen and the detailed design of the structure and its elements undertaken in accordance with the recommendations given in the Codes and Standards given in paragraph 5.2.*

8.1.2 Additional structural provisions

In addition to the robustness strategy from BS EN 1991-1-7, it is advised that all the provisions of robustness that are recommended for Class 2b buildings should also be applied to Class 3 buildings, unless there are specific reasons why they are not appropriate.

8.2 Risk assessment

A systematic risk assessment is the major difference between the Eurocode robustness strategy of Class 3 buildings and that of Class 2b buildings. The purpose of a risk assessment is to determine whether there are any hazard scenarios that have an unacceptable level of risk and if so to suggest steps to mitigate those risks. A reasonable basis for the risk assessment is that the robustness strategy for Class 2b buildings has been applied as a minimum requirement.

Figure B.1 of BS EN 1991-1-7 (reproduced here as Figure 8.1) presents a flow diagram of the overall risk analysis procedure.

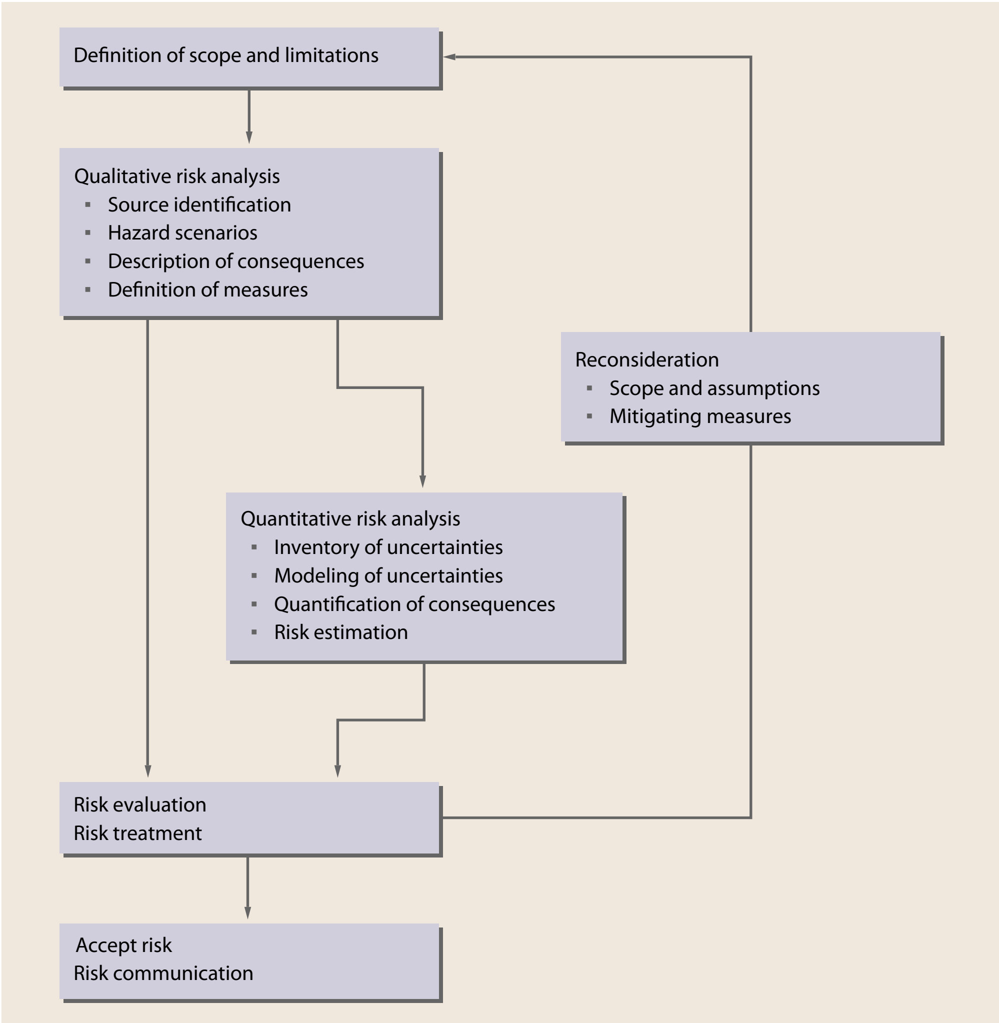


Figure 8.1 Overview of risk analysis (from BS EN 1991-1-7)

Both qualitative and quantitative approaches to risk analysis are presented in Figure 8.1. The risk assessment methodology that is used should be of sufficient detail to enable the hazard related risks to be ranked in order for the subsequent consideration of what risk reduction measures might be required. The rigour of assessment should be proportionate to the complexity of the problem and the magnitude of risks. With the exception of the nuclear and chemical industry, it is unusual for a building to have a quantitative risk assessment.

BS EN 1991-1-7, B.4 implies that a risk analysis for a Class 3 structure should have a descriptive (qualitative) part and, where relevant and practicable, also should have a numerical (quantitative) part.

Guidance on selecting an appropriate risk assessment method is provided by the HSE in Reference 40. Although the HSE document is for offshore installations the principles can be applied to Class 3 buildings. The level of risk assessment should be sufficient to enable the decision making process to be conducted and those responsible for the decision making should be suitably qualified, experienced and of sufficient seniority to be competent.

For 'ordinary' Class 3 buildings (i.e. those that marginally exceed the limits of a Class 2b building) a qualitative risk assessment should generally be used. A quantified assessment might be required for certain hazards if further detail is required to assess the acceptability of the risk. However, a lack of accurate data on the likelihood of hazard events can mean that a quantitative assessment is rarely possible. Therefore, the engineer will be required to apply professional judgement.

Qualitative and quantitative risk assessments can be broken down into the basic steps shown in Figure 8.2. The following sections explain each of these steps in turn.

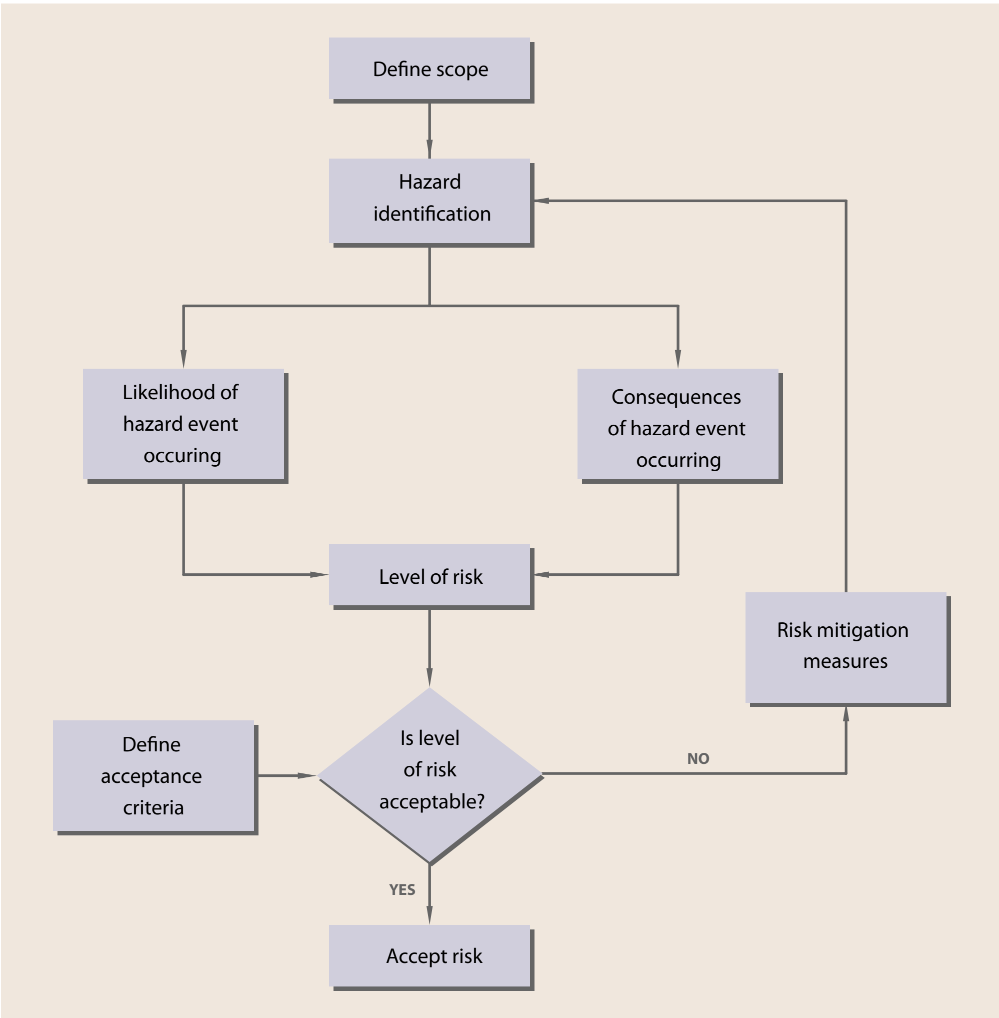


Figure 8.2
Risk assessment process

Note that the terminology used in BS EN 1991-1-7 regarding risks and hazards is different from the terminology that has commonly been used in the UK (e.g. in HSE guidance). In BS EN 1991-1-7, risk is "*a measure of the combination … of the probability or frequency of occurrence of a defined hazard and the magnitude of the consequences of the occurrence*". In contrast, the HSE defines risk as "*the likelihood that a hazard will actually cause its adverse effects, together with a measure of the effect*"[41]. In this publication, the Eurocode terminology has been used.

8.2.1 Defining the scope

The definition of the risk assessment should include the purpose of the risk assessment (e.g. to demonstrate compliance with regulations and any client requirements), the time frame to be considered (e.g. the life of the building) and the types of hazards to be included (e.g. accidental actions). The Building Regulations are intended to guard against accidental events and BS EN 1991-1-7 repeatedly refers to accidental actions. Therefore, it would seem reasonable in most cases to exclude deliberate or malicious hazardous actions from the scope of the risk assessment. However, the designer has an obligation to consider all hazards and foreseeable resultant risks, from whatever source. Occasionally a malicious action might need consideration; this should be discussed with the client. The appropriate robustness strategy will need to be determined on a case by case basis.

If the building is required to be designed to resist malicious actions such as a terrorist attack, this should be addressed and considered in an integrated manner along with the other hazard scenarios. Designing buildings to avoid disproportionate collapse due to accidental actions can provide some robustness against deliberate and malicious actions.

8.2.2 Hazard identification

BS EN 1991-1-7 states that foreseeable and unforeseeable hazards should be considered. Approved Document A states that the risk assessment should include; normal hazards that can reasonably be foreseen and any abnormal hazards. Hazards in general terms are events that have unwanted consequences, which for buildings is structural damage (and the consequential harm).

In practice, the focus of the risk assessment should be on likely hazards and clear mitigation measures that can be taken against them. Below is a list of hazards that should be considered in the risk assessment. Certain buildings might have additional specific hazards that also need to be considered:

- Aircraft impact
- External explosion
- Flooding or extreme tides
- Internal explosion
- Landslide
- Rock fall

- Ship impact
- Train impact
- Vehicle impact
- Fire
- Vandalism
- Extreme weather
- Use beyond original design (e.g. floor overloading).

Other hazards such as design or construction errors and material faults should be addressed by conventional design and construction practice. However, there can be situations where additional measures are justified, e.g. additional testing to ensure adequate material properties.

A structure's sensitivity to changes in design assumption should be considered together with the associated mode of failure. Failure modes should be ductile.

The creation of a hazard register should be the absolute minimum for Class 3 buildings, to demonstrate that the possible hazards have at least been thought about by the designer. Often this should be based on consultation with other experts in a Quality Design Review (QDR) [42].

In addition to the disproportionate collapse considerations, it can be a requirement of certain buildings that they are separately designed to resist specific hazards (e.g. malicious actions). Comprehensive guidance for designing steel buildings to resist terrorist explosions is given in SCI publication P244 [16]. Reference 43 provides specific guidance aimed at improving the safety of tall buildings. Any hazard that has been specifically addressed as a client requirement outside the disproportionate collapse requirements need not be reconsidered in the risk assessment required for Class 3 buildings.

8.2.3 Likelihood of hazard events

Even for a purely qualitative method, the likelihood of each hazard event needs to be estimated and assigned to a predetermined category of likelihood. The number of categories needs to be sufficient to differentiate between hazard events with significantly different probabilities. Five or six categories are usually appropriate for a qualitative risk assessment. Reference 44 suggests the likelihood categories presented in Table 8.1.

LIKELIHOOD	FREQUENCY
Frequent	More than 10 per year
Likely	Between 1 and 10 per year
Occasional	Between 1 every year and 1 every 10 years
Unlikely	Between 1 every 10 years and 1 every 100 years
Rare	Between 1 every 100 years and 1 every 1000 years
Improbable	Between 1 every 1000 years and 1 every 10000 years

Table 8.1
Likelihood categories

For a quantified risk assessment (QRA), various techniques can be used to obtain a probability of occurrence for many of the foreseeable hazard events. Some accidental impacts can be calculated based on historical data. A paper on gas explosions [45] includes useful information (albeit historical) on yearly probabilities of explosions in dwellings in the UK (1 in 500,000 probability of causing structural damage). However, that only considers the risk of one house amongst the UK stock being affected. It does not inform of the risk to any particular house. Overall, a balance has to be struck between expenditure on all houses to fully protect them as against the tolerability of accepting the occasional severe loss.

Some probabilities are so low as to be ignored, even though the consequences can be huge. Thus the HSE publication [46] includes data on the probability of certain hazard events occurring e.g. an aeroplane crashing into an empty football stadium in the UK is quoted as 1 in a million per annum. The probability of crashing into a full stadium is even less; the probability is so low it is customarily ignored. It is impossible to come up with a meaningful prediction of terrorist attack or of the form that attack might take; it is simply a judgement.

Carrying out QRA for any particular project is possible for many hazards but is best left to specialists.

8.2.4 Consequences of hazard events

For a qualitative analysis method the consequences of hazard events are dealt with in a similar manner to the likelihood of hazard events in that they are assigned to a category reflecting their severity. The number of categories can vary but five or six categories are usually appropriate for a qualitative risk assessment. Engineering judgement, experience and approximate calculations can be used to estimate the consequences for a qualitative assessment. Reference 44 suggests the severity categories presented in Table 8.2.

For a quantified risk assessment the consequences can be measured in various units. The amount of structural damage is often the most appropriate measure for buildings; alternatively an estimate of the number of casualties can be used. It can be appropriate to consider human consequences and structural consequences separately. Structural calculations will be necessary to determine the amount of damage caused.

Table 8.2
Severity categories

SEVERITY	CONSEQUENCES
Disastrous	20% to 100% collapse
Extreme	15% collapse of floor to 20% collapse of building
Serious	Up to 15% collapse of floor
Significant	Loss of structural member local to event but no floor collapse
Minor	Local structural damage but no loss of structural members
Negligible	No structural damage

Guidance regarding the calculation of impact forces and explosion loads due to various causes is given in Annexes C and D of BS EN 1991-1-7.

The performance of the damaged structure then needs to be assessed to determine if further collapse will occur. The sensitivity of the building to variations in design assumptions should be considered.

It must be kept in mind that the Building Regulations only require that buildings are designed to avoid disproportionate collapse not designed to survive all possible events. Determining whether collapse is disproportionate is not a straightforward issue. Guidance given in Approved Document A suggests that for the notional removal of one column, damage not exceeding 70 m^2 or 15% of the floor area (whichever is less) is proportionate. Note: The limit is 100 m^2 or 15% in BS EN 1991-1-7. However, because the damage should not be disproportionate to the cause, the amount of damage that is acceptable is related to the size of the original accidental action. Hence, the larger the original cause the more collapse becomes acceptable in terms of disproportionate collapse. If the initial event is large enough, total collapse of a building may not be considered disproportionate.

8.2.5 Level of risk

The level of risk associated with each hazard is usually expressed as a function of the severity and the likelihood of the hazard event.

For a qualitative assessment, a risk matrix as shown in Figure 8.3 is a convenient method of ranking the risks. Each hazard event is plotted on the risk matrix according to the appropriate severity and likelihood category. The acceptability of risks should be evaluated in order, starting with the highest risk.

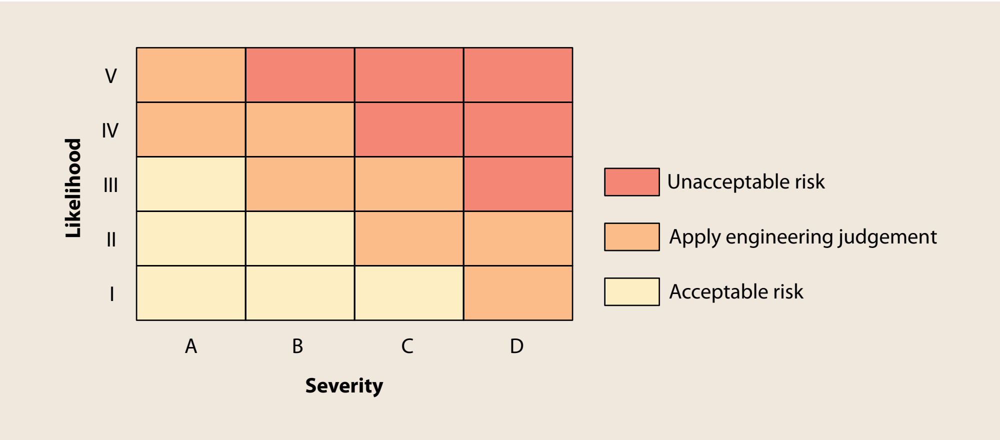


Figure 8.3 Risk matrix for hazard events

Where the likelihood is categorised by events that are more or less likely to occur within the design life of the building, and the severity of damage is assessed as being more or less than the collapse of 15% of a floor (the notional limit given in Approved Document A and BS EN 1991-1-7), it is often possible to simplify the considerations into a simple 2 by 2 matrix (as shown in Figure 8.4). Two boxes in this matrix require the application of engineering judgement. As examples of these, a Class 3 building being built near a steep slope might be expected to experience minor rock falls occasionally within its design life but a major landslide might be expected no more frequently than, say, once in 500 years (well beyond the design life). The latter event, although rare, could have extreme or disastrous consequences, whereas the former event, while occasional, would have far less serious consequences. In the latter case it might be assessed that, by apply structural tying as recommended for Class 2b buildings, the structure was sufficiently robust. For the former event, tying alone would not be sufficient as the whole structure could be subjected to overturning in a landslide. It would generally be unrealistic to design the structure to resist the landslide event. Hence, depending on the nature of the building use and occupancy, the risk could be accepted, reduced (e.g. by placing a heavy diversionary structure between the building and the slope) or removed (e.g. by stabilising the slope or relocating the building).

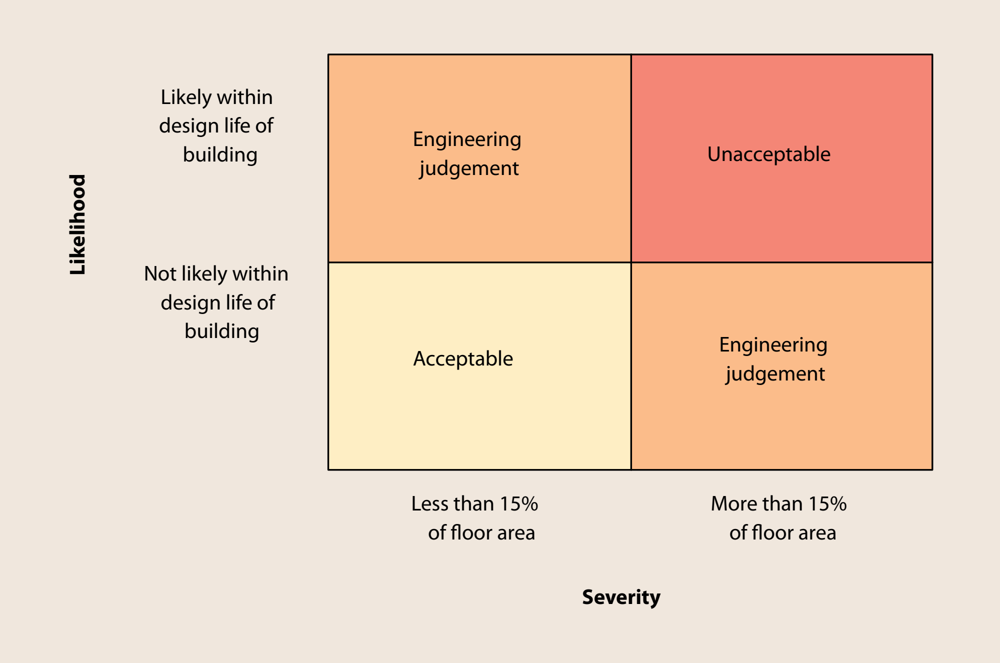


Figure 8.4
Simplified risk matrix

8.2.6 Acceptance criteria

The acceptability criteria of risk for each building should be agreed with the building control authorities and the client. Indicative information on what levels of risk can be acceptable can be obtained from various sources. For example, the nuclear [46] and the offshore [40] industries have guidance regarding acceptable levels of risk but these are generally expressed in terms of risk of death per individual per year. Reference 47 discusses some of the issues concerning acceptability of risk in structural engineering.

BS EN 1991-1-7 presents the risk acceptance principle of ALARP (as low as reasonably practicable). According to this principle, two risk levels are specified: if the risk is below the lower bound of the broadly tolerable (i.e. ALARP) region no measures need to be taken; if it is above the upper bound of the broadly tolerable region the risk is considered as unacceptable. If the risk is between the upper and lower bound an economical optimal solution should be sought.

BS EN 1991-1-7 suggests that risk acceptance criteria should be based on the following two criteria:

- The individual acceptable level of risk. Individual risks are usually expressed as fatal accident rates. They can be expressed as an annual fatality probability or as the probability per time unit of a single fatality when involved in a specific activity.
- The socially acceptable level of risk. The social acceptance of risk to human life, which can vary with time, is often presented as an F-N curve, indicating a maximum yearly probability F of having an accident with more then N casualties.

In reality, for many risks it will be an informed judgement.

8.2.7 Risk mitigation

Risks can be mitigated in one of two ways:

- By eliminating the hazard event.
- By reducing the probability of the hazard event and/or the severity of the consequences.

Totally eliminating the hazard event is not possible or practical in many situations but significantly reducing the probability or the consequences is often achievable at very little additional cost.

Eliminating hazards

The overall building concept can have a significant influence on the type and magnitude of hazards that need to be addressed. This includes the building location and proximity to specific hazards. The building structural form must also be considered. Large parts of the building should not be reliant on one or two critical members. Where possible, loads should be distributed between many members and alternative load paths should be present which could be utilised in the event of an incident.

Some hazards can be avoided. Deliberate or accidental vehicular impact on the building could be prevented by the installation of suitable external barriers. Excluding explosive materials from a building will avoid the hazard of their explosion.

Reducing the probability of hazard events

Reducing the probability of hazard events will often be beyond the control of the structural engineer for a particular building. For Class 3 buildings, a review of the proposed design should be carried out that specifically focuses on robustness and reliability. Part of that review is to consider the possibility and implications of error.

Not all risk reduction measures will involve a structural solution. A simple but effective method of reducing the likelihood of terrorist attack is to have security checks on people entering the building.

Reducing the consequences of hazard events

There are many measures that can be adopted to reduce the consequences of hazard events. Providing increased levels of robustness (e.g. providing reserves of strength, alternative load paths, and resistance to degradation) is the most obvious.

Introducing ductility into the structural system is a means of reducing the consequences of the hazards. Ductility is the standard demand in earthquake protection and it is often achieved by increasing the joint resistances so that they are stronger than the members. Strengthening the members but not the connections might enforce an undesirable brittle failure under extreme loading if the connection failure mode is not ductile.

Sub-dividing larger buildings with movement joints can be used to restrict the spread of collapse. Sprinklers can be installed to control the spread of fire and venting panels can be installed to reduce the blast loading from explosions. Traffic calming measures can be used to reduce the speed of accidental vehicular impact.

Failure of beams supporting one or more columns or systems providing lateral stability is likely to have particularly severe consequences and the tying design method could prove inadequate in this particular situation. It is recommended that either element removal or key element design is used.

8.2.8 Risk acceptance and communication

The final stage of the systematic risk assessment is to accept the residual risks and report the findings. The report should include all the hazards and their associated level of risk, with explanations of the basis on which the risks are considered acceptable and describing the reduction measures that have been adopted to achieve acceptable levels. Optional additional reduction measures can also be suggested to further reduce the risks. The findings of the risk assessment will feed back into the decision-making process for the design and operation of the building. All the sources of data, assumptions and uncertainties in the assessment should be included in the report.

TRANSFER BEAMS

9.1 General

A transfer beam is a beam that supports one or more columns, as shown in Figure 9.1. From basic engineering it is clear that a transfer beam is more critical than a floor beam and potentially more critical than a column member, simply by examining the floor area that is dependent on the transfer beam. Therefore, when designing structures to resist accidental actions, transfer beams, their connections and the members that support them need careful consideration.

In view of their potential criticality, all transfer beams, in whatever Class of structure, should be subject to an assessment to determine whether the standard approaches of Class 1, 2a or 2b are appropriate. Class 3 requires such an assessment in any event.

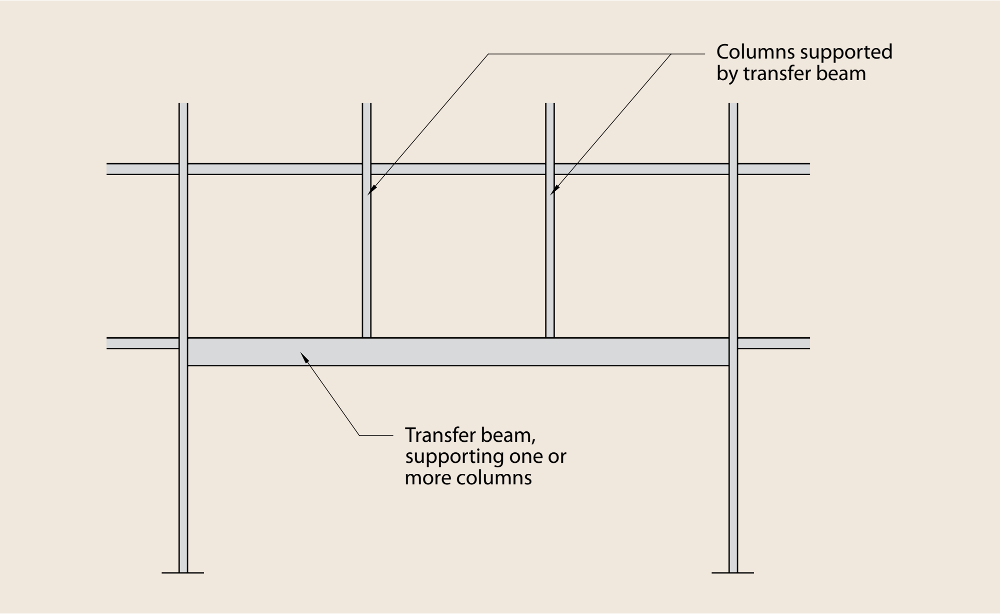


Figure 9.1
Transfer beam

9.2 Class 1 buildings

In BS EN 1991-1-7 there is no specific mention of transfer beams in relation to Class 1 buildings. Therefore, it is recommended that transfer beams in Class 1 buildings should be designed to satisfy the minimum level of horizontal tying stated in Section 5.1.2, i.e. the transfer beam and its end connections should be designed to resist a minimum design tensile force of 75 kN.

9.3 Class 2a buildings

There is also no specific mention in BS EN 1991-1-7 of transfer beams in relation to Class 2a buildings. Therefore, it is recommended that transfer beams in Class 2a buildings should be designed to satisfy the specified minimum level of horizontal tying for Class 2a buildings.

Transfer beams, including their end connections, should be capable of sustaining a design tensile load of "T_i" for internal beams, and "T_p", in the case of perimeter beams. The magnitudes of "T_i" and "T_p" are calculated according to Equations A.1 and A.2 from BS EN 1991-1-7, A.5.1, with the addition of the terms V_c to represent the load from supported columns, i.e.:

$$T_i = 0.8(g_k + \psi q_k)sL + 0.5V_c$$ or 75 kN, whichever is the greater

$$T_p = 0.4(g_k + \psi q_k)sL + 0.5V_c$$ or 75 kN, whichever is the greater

In which s and L are the spacing and length of the transfer beam and V_c is the sum of the point loads from the columns supported by the transfer beam. The V_c term is calculated for the accidental design case, as given in Section 7.6.5. (See Section 6.2.2 for the definition of other terms.)

The loss of a transfer beam will generally mean the loss of a substantial part of a building so, irrespective of a pure interpretation of BS EN 1991-1-7, sound engineering would suggest that robust connections should be provided to transfer beams. The end connections should generally have a tying resistance approaching the full shear resistance of the transfer beam.

9.4 Class 2b buildings

As explained in Section 7, there are three alternative methods for designing Class 2b buildings for robustness; tying, notional removal and key element. It is possible to include transfer beams within any of these methods. However, the tying or key element approaches are usually the most appropriate.

9.4.1 Notional removal

Transfer beams are specifically mentioned for Class 2b buildings in BS EN 1991-1-7 in the notional removal approach. However, as discussed in Section 7.6.4, the notional removal approach is unlikely to be a viable design strategy for most steel frame buildings, even less so where transfer beams are used.

9.4.2 Tying approach

The tying approach presented in BS EN 1991-1-7 does not give any specific requirements for transfer beams. The general requirements of the tying approach are presented in Sections 7.2 to 7.5. For transfer beams the horizontal and vertical tying requirements should be modified from the general requirements.

The horizontal tying requirement should be modified and applied as described in Section 9.3.

The vertical tying requirements should be modified so that columns supported by transfer beams are vertically tied to the transfer beam. The tensile resistance of the connection should be designed for the maximum tension that would occur in the accidental loading case if either one of the columns supporting the transfer beam was removed. The intention of this requirement is that the transfer beam could, in part, be supported from the structure above if one of its supporting columns were lost due to an accidental event (see Figure 9.2).

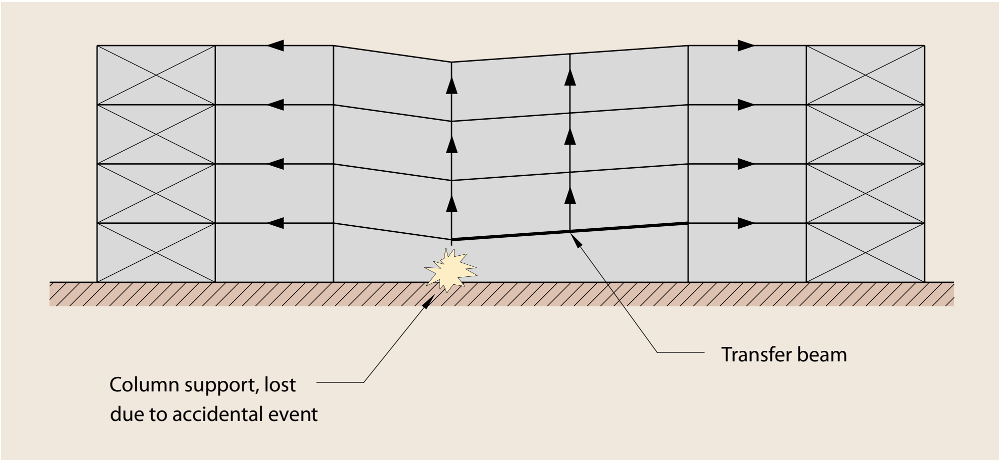


Figure 9.2
Transfer beam supported from above

9.4.3 Key element approach

BS EN 1991-1-7 does not require transfer beams in Class 2b buildings to be designed as key elements but transfer beams may be designed as key elements instead of applying the tying method (as modified in Section 9.4.2) or the notional removal method.

The general guidance on the design of key elements given in Section 7.7 should be applied to key element transfer beams.

In addition to designing the transfer beam as a key element, the columns that support the transfer beams should also be designed as key elements.

9.5 Class 3 buildings

As a minimum requirement, transfer beams in Class 3 buildings should be designed following the guidance for transfer beams in Class 2b buildings (see Section 9.4). However, it must be recognised that in some Class 3 buildings the loss of a transfer beam could be catastrophic which would justify more stringent robustness provisions. The risk assessment process for a Class 3 building will identify whether any additional measures are needed with regard to the transfer beam.

SUMMARY OF ROBUSTNESS REQUIREMENTS

Table 10.1 provides a summary of the robustness requirements for the different building classes. The requirements are divided into two categories, the requirements of the Eurocodes and those requirements that are recommended in addition to the Eurocode requirements. Detailed explanations of all the requirements are given in the previous sections of this publication.

ROBUSTNESS REQUIREMENTS	BUILDING CLASS					
	1	2a	2b T	2b NR	2b KE	3
Eurocode						
No additional robustness requirements if designed to BS EN 1993	■					
Provide horizontal ties		■	■			
Provide vertical tying			■			
Notional removal method				■		
Key element method					■	
Systematic risk assessment						■
Recommended additional						
Minimum horizontal tying resistance of 75 kN for beam to column connections	■					
Bearing details for floor, roof and stair units should conform to BS EN 1992 and include allow for tolerances		■	■	■	■	■
Multiple bracing systems			■	■	■	■
Anchorage of heavy floor/roof/stair units			■	■	■	■
Apply rules for class 2b as a minimum						■

Table 10.1 Summary of robustness requirements

Note:
2b T = Tying design method
2b NR = Notional removal design method
2b KE = Key element design method

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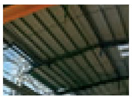

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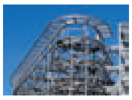

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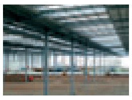

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APPENDIX A

Six short worked examples are presented:

Example 1 – Class 1 building

Brief

What are the robustness requirements for the building shown in outline in Figure 1?

The building is a single storey agricultural building of portal frame construction. It is 15 m wide, 30 m long and height to eaves is 6 m. The cladding weight is less than 0.7 kN/m².

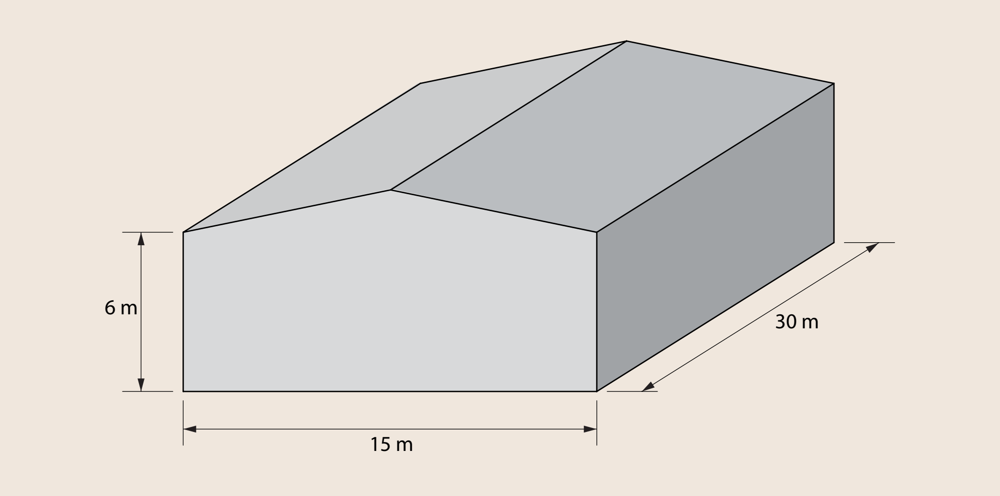

Figure 1
Agricultural building

Building classification

BS EN 1991-1-7, Table A.1

All agricultural buildings are classified as Class 1. (Although agricultural buildings are not covered by UK Building Regulations, they are classified in BS EN 1991-1-7.)

Robustness requirements

Robustness strategy

Section 5.1.1

The building is designed in accordance with the rules given in BS EN 1993. Therefore no additional rules need to be applied for the consideration of avoidance of disproportionate collapse.

Additional structural provisions

Section 5.1.2

It is recommended that a minimum level of horizontal tying is provided within the frame. The recommended minimum level of horizontal tying is that all floor beam-to-column connections are designed to be capable of sustaining a design tensile force of 75 kN.

Section 5.2.2

The roof only supports roof cladding that weighs not more than 0.7 kN/m² and carries only imposed roof loads and wind loads. Therefore, the minimum level of horizontal tying should also be applied to roof beams.

Taking the above into account, the connections between columns and roof beams (rafters) should be capable of sustaining a design tensile force of 75 kN.

Example 2 – Class 2a building

Brief

What are the robustness requirements for the steel-framed building with the beam and column configuration shown in Figure 1?

The building is a three storey hotel of braced frame construction. The columns are laid out on a 7.5 m x 7.5 m grid with ASBs spanning in one direction and inverted T sections connecting the columns in the orthogonal direction. The ASBs support composite slabs formed from in-situ concrete and deep decking supported on the bottom flange of the ASBs.

The floor loading is:

Permanent action, $g_k = 4.0 \text{ kN/m}^2$

Variable action, $q_k = 3.5 \text{ kN/m}^2$

The roof loading is:

Permanent action, $g_k = 4.0 \text{ kN/m}^2$

Variable action, $q_k = 1.0 \text{ kN/m}^2$

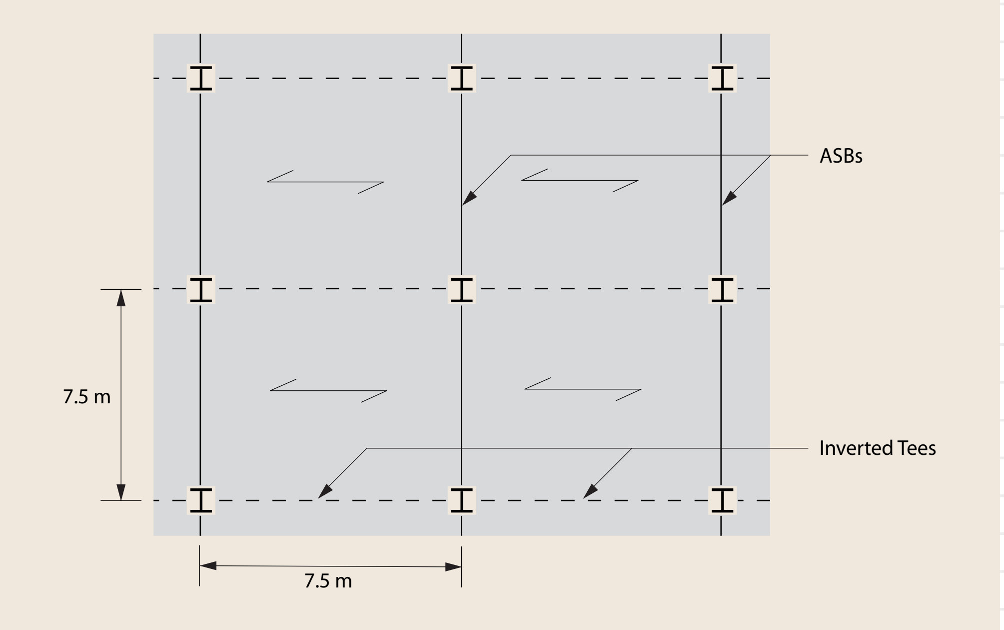

Figure 1
Floor arrangement

Building classification

BS EN 1991-1-7, Table A.1

A hotel not exceeding four storeys is classified as Class 2a.

Robustness requirements

Robustness strategy

Section 6.1.1

The building is Class 2a therefore effective horizontal ties should be provided.

Section 6.2.3

Effective horizontal ties should be provided in the following locations:

- around the perimeter of each floor level;
- around the perimeter of each roof level;
- internally in two right angle directions to tie columns;
- all beams designed to act as ties.

Section 6.2.2 The required tie resistances are given by the following equations:

T_i	$= 0.8(g_k + \psi q_k) s L$	or 75 kN, whichever is the greater
T_p	$= 0.4(g_k + \psi q_k) s L$	or 75 kN, whichever is the greater

where:

g_k	is the permanent action
q_k	is the variable action
s	is the spacing of ties
L	is the span of the tie
ψ	is the relevant factor ψ_1 or ψ_2. According to the NA to BS EN 1990, ψ_1 should be used and, for hotels, Category C: congregation areas applies, so $\psi_1 = 0.7$.

Floor internal ASB (perpendicular to span)

Section 6.3.8

T_i	$= 0.8(g_k + \psi_1 q_k) s L$	or 75 kN, whichever is the greater
T_i	$= 0.8(4.0 + 0.7 \times 3.5)\ 7.5 \times 7.5$	
T_i	$= 290$ kN	

The member, including its end connections, should be capable of sustaining a design tensile load of 290 kN.

Floor internal tee (parallel to span)

Section 6.3.8 The internal tee section parallel to span does not carry any floor loads. Therefore, the tie force equations do not need to be considered and the minimum tie force of 75 kN should apply for this member.

The member, including its end connections, should be capable of sustaining a design tensile load of 75 kN.

Floor edge beam (perpendicular to span)

Section 6.3.8
Figure 6.11

T_p	$= T_3 = 0.4(g_k + \psi_1 q_k) s L$	or 75 kN, whichever is the greater
T_p	$= 0.4(4.0 + 0.7 \times 3.5)\ 7.5 \times 7.5$	
T_p	$= 145$ kN	

The member, including its end connections, should be capable of sustaining a design tensile load of 145 kN.

Floor edge beam (parallel to span)

Section 6.3.8 The member does not support any floor loads. Therefore, the tie force equations do not need to be applied and the minimum tie force of 75 kN should be applied to this member.

The member, including its end connections, should be capable of sustaining a design tensile load of 75 kN.

Roof internal beam (perpendicular to span)

Section 6.3.8 The value of ψ_1 taken from Table NA.A1.1 in the NA to BS EN 1990 appropriate to roofs applies, so $\psi_1 = 0$.

$T_i \quad = 0.8(g_k + \psi q_k)sL$ or 75 kN, whichever is the greater

$T_i \quad = 0.8(4.0 + 0 \times 1.0)7.5 \times 7.5$

$T_i \quad = 180 \text{ kN}$

The member, including its end connections, should be capable of sustaining a design tensile load of 180 kN

Roof internal tee (parallel to span)

Section 6.3.8 The internal tee section parallel to span does not carry any roof loads. Therefore, the tie force equations do not need to be applied and the minimum tie force of 75 kN should be applied to this member.

The member, including its end connections, should be capable of sustaining a design tensile load of 75.0 kN.

Roof edge beam (perpendicular to span)

$T_i \quad = 0.4(g_k + \psi q_k)sL$ or 75 kN, whichever is the greater

$T_i \quad = 0.4(4.0 + 0 \times 1.0)7.5 \times 7.5$

$T_i \quad = 90 \text{ kN}$

The member, including its end connections, should be capable of sustaining a design tensile load of 90 kN.

Roof edge beam (parallel to span)

The member does not support any roof loads. Therefore, the tie force equations do not need to be considered and the minimum tie force of 75 kN should apply for this member.

The member, including its end connections, should be capable of sustaining a design tensile load of 75 kN.

Additional structural provisions

The bearing details of the floor and roof slabs should conform to BS EN 1992 and make due allowance for construction, fabrication and manufacturing tolerances.

The floor is formed from deep decking supported on the bottom flange of ASBs and in situ concrete, see Figure 2. Therefore, the slabs and beams are integrated and the bearing details are not dependent on construction, fabrication and manufacturing tolerances.

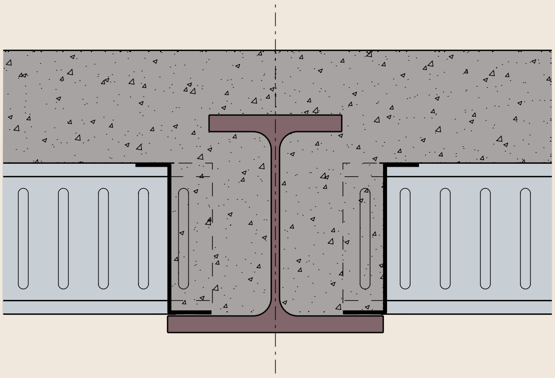

Figure 2
Slab bearing detail

Example 3 – Class 2b building – Tying method

Brief

What are the robustness requirements for the steel-framed multi-storey building outlined in Figure 1?

The building is a 10 storey office building of steel braced frame construction. All storeys are 4.0 m high, apart from the ground to first floor, which has a height of 5.0 m. The columns are laid out on a 6 m × 9 m grid with the primary beams spanning 6 m and the secondary beams spanning 9 m, as shown in Figure 2. The spacing of the secondary beams is 3.0 m. A composite flooring system is used with steel decking spanning between the secondary beams. All the secondary and primary beams are assumed to act compositely with the floor slab. The steel frame has two braced bays on each of the four sides providing lateral stability.

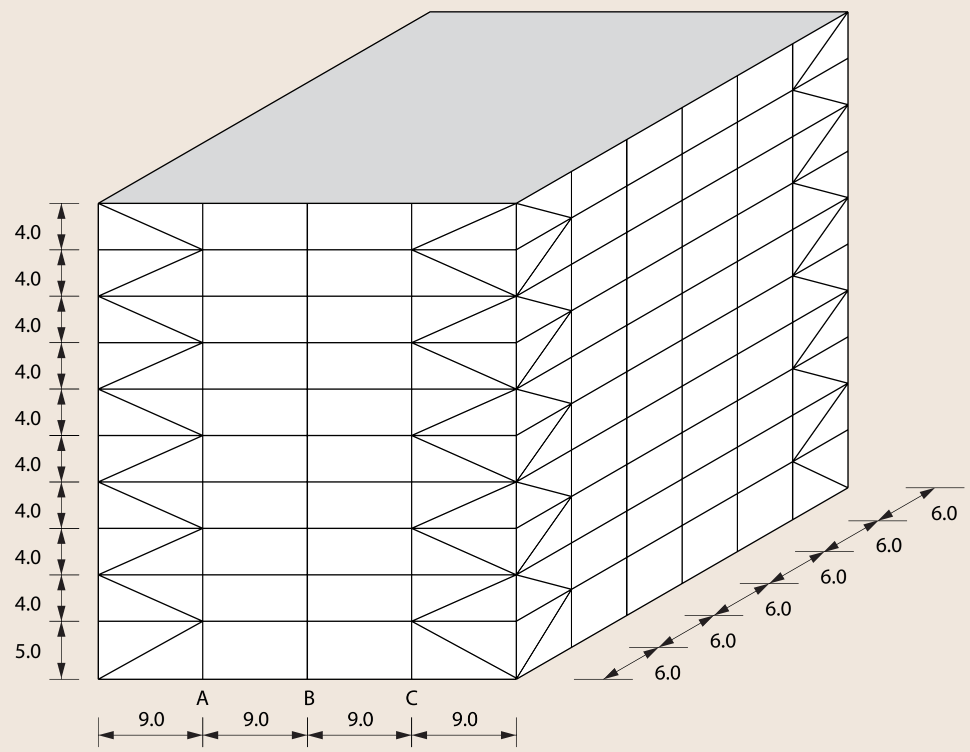

Figure 1
Office building

In practice, the robustness checks must be carried out on all members to ensure adequate robustness throughout the structure. However, in this example, the checks are only performed on internal floor beams (primary and secondary), an edge column and an internal column. These columns are denoted B and E respectively in Figure 2.

The composite floor system comprises steel decking spanning between the secondary beams, as shown in Figure 3, with a 125 mm thick slab in grade C25/30 concrete.

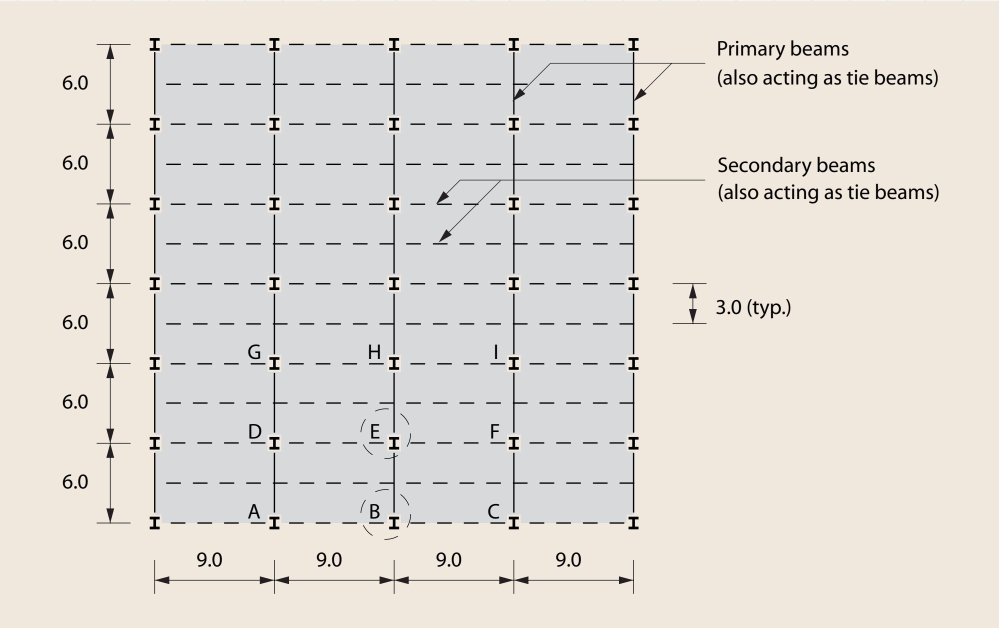

Figure 2
Beam arrangement

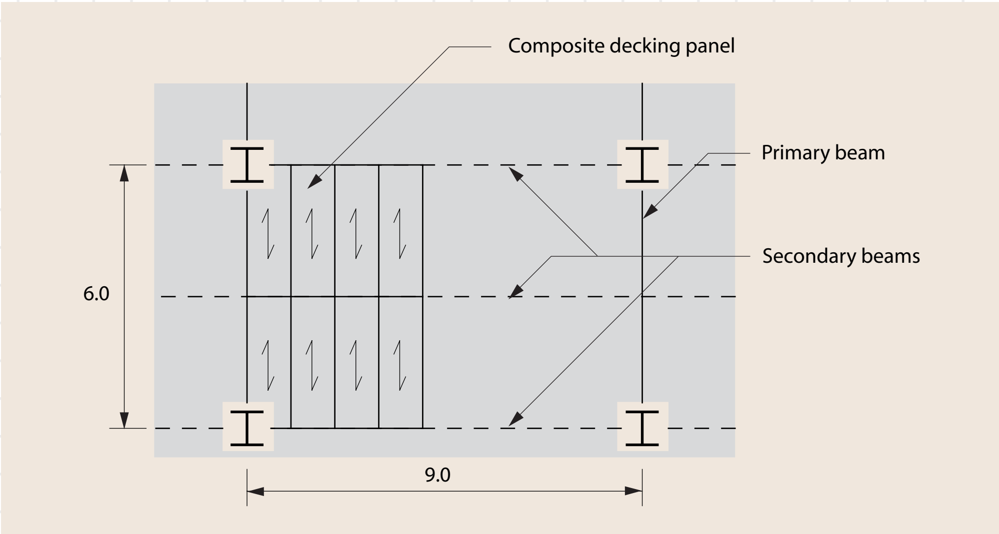

Figure 3
Slab arrangement

The floor loading is:

Permanent action, $g_k = 3.5\ \text{kN/m}^2$

Variable action, $q_k = 6.0\ \text{kN/m}^2$

Building classification

BS EN 1991-1-7 Table A.1

Office buildings greater than four storeys but not exceeding 15 storeys are classified as Class 2b.

Robustness requirements

Robustness strategy

Section 6.1.1

The building is classified as Class 2b, therefore there are three alternative methods that could be applied:

- Tying.
- Notional removal.
- Key element.

For this example the tying method will be applied. For the tying method the following should be provided:

- Horizontal ties.
- Vertical ties.

Horizontal ties

Section 6.2.3

The required horizontal tie resistances are given by the following equations:

$T_i = 0.8(g_k + \psi q_k) s L$ or 75 kN, whichever is the greater

$T_p = 0.4(g_k + \psi q_k) s L$ or 75 kN, whichever is the greater

where:

g_k is the permanent action

q_k is the variable action

s is the spacing of ties

L is the span of the tie

ψ is the relevant factor ψ_1 or ψ_2. According to the NA to BS EN 1990, ψ_1 should be used and, for offices, Category B: office areas applies, so $\psi_1 = 0.5$.

Floor internal primary floor beam

Section 6.3.9

$T_i = 0.8(g_k + \psi q_k) s L$ or 75 kN, whichever is the greater

$T_i = 0.8(3.5 + 0.5 \times 6.0)\ 9.0 \times 6.0$

$T_i = 281 \text{ kN}$

The member, including its end connections, should be capable of sustaining a design tensile load of 281 kN.

Floor internal secondary floor beam

Section 6.3.9

$T_i = 0.8(g_k + \psi q_k) s L$ or 75 kN, whichever is the greater

$T_i = 0.8(3.5 + 0.5 \times 6.0)\ 3.0 \times 9.0$

$T_i = 140 \text{ kN}$

The member, including its end connections, should be capable of sustaining a design tensile load of 140 kN.

Vertical ties

Section 7.3.2

Vertical ties should be:

- provided in columns such that each column is tied continuously from the foundations to the roof level;
- capable of resisting an accidental design tensile force equal to the largest design vertical permanent and variable load reaction applied to the column from any one storey. Such accidental design loading should not be assumed to act simultaneously with permanent and variable actions that may be acting on the structure.

Internal column

The required tie resistance for an internal column splice is given by the following equation:

BS EN 1990, Eq. 6.10b
SCI-P362 Appendix C

$$T_v = (\xi \gamma_g g_k + \gamma_q q_k) s L$$

where:

g_k is the permanent action

q_k is the variable action

s is the column spacing (direction 1)

L is the column spacing (direction 2)

ξ is a reduction factor for unfavourable permanent actions, $\xi = 0.925$

γ_g is a partial factor for permanent actions, $\gamma_g = 1.35$

γ_q is a partial factor for variable actions, $\gamma_q = 1.5$.

Therefore,

$$T_v = (\xi \gamma_g g_k + \gamma_q q_k) s L$$

$$T_v = (0.925 \times 1.35 \times 3.5 + 1.5 \times 6.0) 6.0 \times 9.0$$

$$T_v = 722 \text{ kN}$$

The column splices should be capable of sustaining a design tensile load of 722 kN.

Edge column

The required tie resistance for an edge column splice is given by the following equation:

$$T_v = (\xi \gamma_g g_k + \gamma_q q_k) s L/2$$

$$T_v = (0.925 \times 1.35 \times 3.5 + 1.5 \times 6.0)\ 6.0 \times 9.0/2$$

$$T_v = 361 \text{ kN}$$

The column splices should be capable of sustaining a design tensile load of 361 kN.

Additional structural provisions

Section 7.1.2

The recommended additional structural provisions are concerned with:

- Vertical bracing.
- Anchorage of heavy units.

Vertical bracing

The braced bays should be distributed throughout the building such that, in each of two directions approximately at right angles, no substantial portion of the building is connected to only one system for resisting horizontal force.

In this Example, the vertical bracing requirement is satisfied by having two braced bays on each of the four sides.

Anchorage of heavy units

The floor and roof slabs should be effectively anchored in the direction of their span, either to adjacent slabs over a support, or directly to their supports.

Section 7.5.2

The anchorage should be capable of carrying the self-weight of the floor or roof unit.

The composite floor slabs span 3 m between supports. For this Example it is assumed that their dead weight is equal to the floor permanent action, $g_k = 3.5$ kN/m^2. The required anchorage at each slab support is calculated below.

Required anchorage $= 3.5 \times 3/2 = 5.25$ kN per m width.

Using a properly anchored A142 mesh reinforcement (cross section area of 142 mm^2/m and yield strength of 500 N/mm^2) will provide a tensile anchorage resistance of 71 kN per m width.

BS EN 1992-1-1, Table 2.1N and Table NA.1

Note: For robustness, the material partial factor for the accidental design situation may be used which is 1.0 for reinforcing steel.

For the slabs spanning onto edge beams a suitable detail will be need to anchor the floor slab directly to the edge beam.

If edge beams are designed as composite with the use of welded shear studs, the anchorage requirements will generally be satisfied.

If edge beams are designed as non-composite, welded shear studs may be used to provide the required anchorage, even though the edge beam is non-composite. Alternatively, the shot fired pins that are used to connect the deck to the supporting steelwork may be utilised. The resistance of a shot fired pin connection is dependent on the fixing used and decking thickness. For a shot fired pin connection with a resistance of 2.7 kN, sufficient anchorage is achieved for this Example with shot fired pins spaced at 500 mm along the beam.

Example 4 – Class 2b building – Notional removal method

Brief

This example uses the same building and floor arrangement as described in Example 3. Instead of applying the tying method to design for avoidance of disproportionate collapse, this example considers whether the notional removal method can be applied to this Class 2b building.

Robustness requirements

Robustness strategy

Section 7.1.1 The building should be checked to ensure that upon the notional removal of each supporting column and each beam supporting a column (one at a time in each storey of the building) the building remains stable and that any local damage does not exceed a certain limit.

Figure 7.2 The limit of admissible local damage according to BS EN 1991-1-7, A.4 is 15% of the floor area or 100 m^2, whichever is smaller.

Ref.12 Note: For compliance with Approved Document A the lower limit of 70 m^2 should be applied.

In this example there are no beams supporting columns so only the notional removal of column sections needs to be considered. In practice, the notional removal checks must be carried out on all columns to ensure adequate robustness throughout the structure. However, in this example, the checks are only performed on an edge column and an internal column. These columns are denoted B and E respectively in Figure 1.

It is assumed that the floor slab does not have any resistance to act as a cantilever or to hold up beams designed to support the slab.

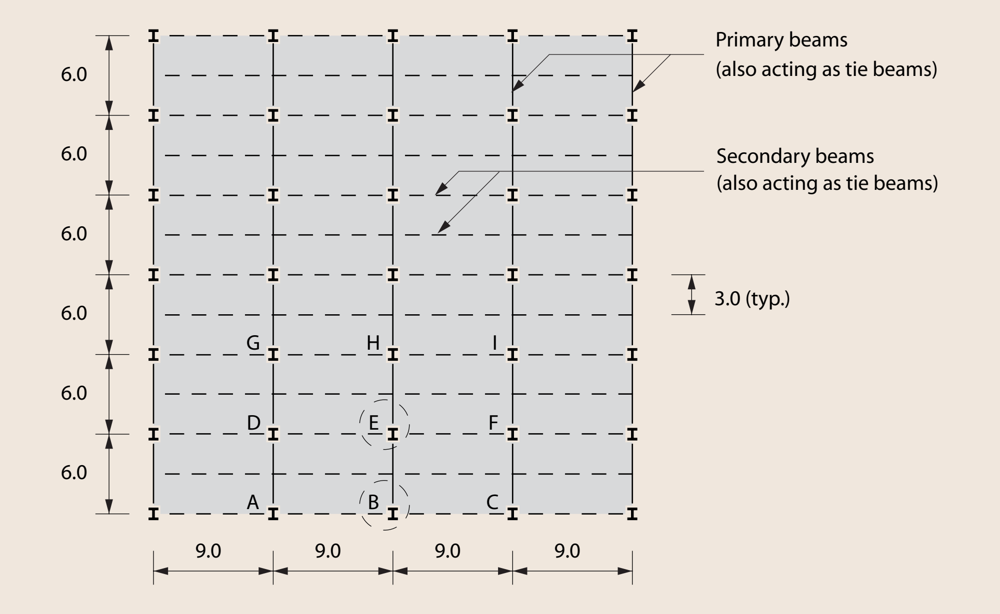

Figure 1
Beam arrangement

Internal column

If column E were notionally removed, the primary beams BE and EH would collapse, leading to the collapse of the secondary beams they support (see Figure 2).

The total floor area that would collapse if column E were notionally removed $= 9 \times 2 \times 6 \times 2 = 216\ \mathrm{m}^2$, which exceeds the limit of 100 m².

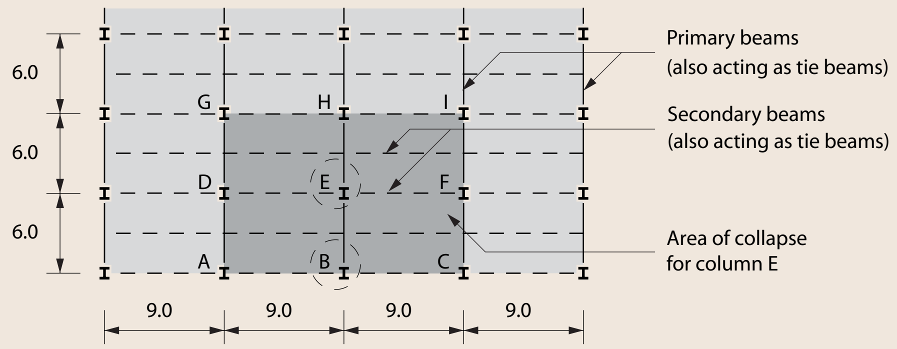

Figure 2
Column E area of collapse

The area of collapse could be even greater than calculated, as the calculated value does not include the possible collapse of floor area on the storeys above where the column section is notionally removed.

Therefore, the notional removal approach cannot be used for internal columns in this building. The tying method or the key element design method should be applied to design for avoidance of disproportionate collapse (see Examples 3 and 5).

Edge column

If edge column B were notionally removed, the edge beams AB and BC would collapse and primary beam BE would collapse, leading to the collapse of the secondary beams it supports (see Figure 3). The total floor area that would collapse if column B were notionally removed $= 9 \times 2 \times 6 = 108\ \mathrm{m}^2$, which exceeds the limit of 100 m².

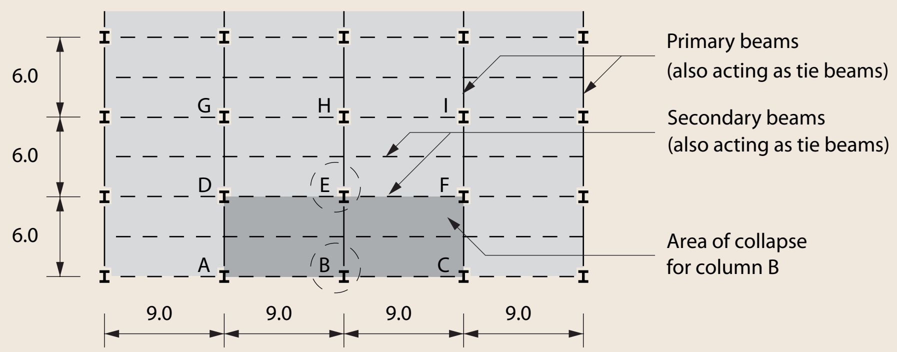

Figure 3
Column B area of collapse

The area of collapse could be even greater than calculated as the calculated value does not include the possible collapse of floor area on the storeys above where the column section is notionally removed.

Therefore, the notional removal approach cannot be used for edge columns in this building. The tying method or the key element design method should be applied to design for avoidance of disproportionate collapse (see Examples 3 and 5).

Example 5 – Class 2b building – Key element method

Brief

This example uses the same 10 storey building and floor arrangement (9 m × 6 m column grid) as described in Example 3. In Example 4 it was shown that the notional removal method cannot be applied to this building (as an alternative to the tying method for avoidance of disproportionate collapse). This example therefore considers the key element method as an alternative to the tying method for avoidance of disproportionate collapse.

All storeys are 4.0 m high, apart from the ground to first floor, which has a height of 5.0 m. Non load-bearing partitioning is constructed between columns and has a blast resistance of 2.0 kN/m². The columns from ground to first floor have a cross-section of 300 mm × 300 mm. The total weight of an internal column for the full height of the building is 50 kN.

The loading is:

Floor: Permanent action,	$g_k = 3.5$ kN/m^2
Floor: Variable action,	$q_k = 6.0$ kN/m^2
Roof: Permanent action,	$g_k = 3.5$ kN/m^2
Roof: Variable action,	$q_k = 1.0$ kN/m^2

Robustness requirements

Robustness strategy

Section 7.1.1

Where the notional removal of columns or beams supporting columns would result in an extent of damage in excess of the limit (which was shown in Example 4), such elements should be designed as key elements.

Section 7.7.2

BS EN 1991-1-7, A.8

Key elements should be capable of sustaining an accidental design action of A_d applied in horizontal and vertical directions (in one direction at a time) to the member and any attached components. The recommended value of A_d for building structures is 34 kN/m².

In this example there are no beams supporting columns, so only the key element design of column sections needs to be considered. In practice, the key element design checks must be carried out on all columns to ensure adequate robustness throughout the structure. However, in this example, the checks are only performed on an internal column.

Two loading cases will be considered in this example:

- Maximum accidental blast load over a partial width of wall.
- Reduced accidental blast load over a fully loaded width of wall.

Internal column

The area to which the accidental loading is applied is dependent on what is attached to the key element and, in particular, its integrity under blast loading. In this example there is partitioning running between columns. As the partitioning is not load-bearing, it is reasonable to assume that it is mostly blown out by the blast, leaving only a small section attached to the key element as shown in Figure 1. In this case, the breadth of partitioning remaining after the blast is estimated to be $b + 200$ mm.

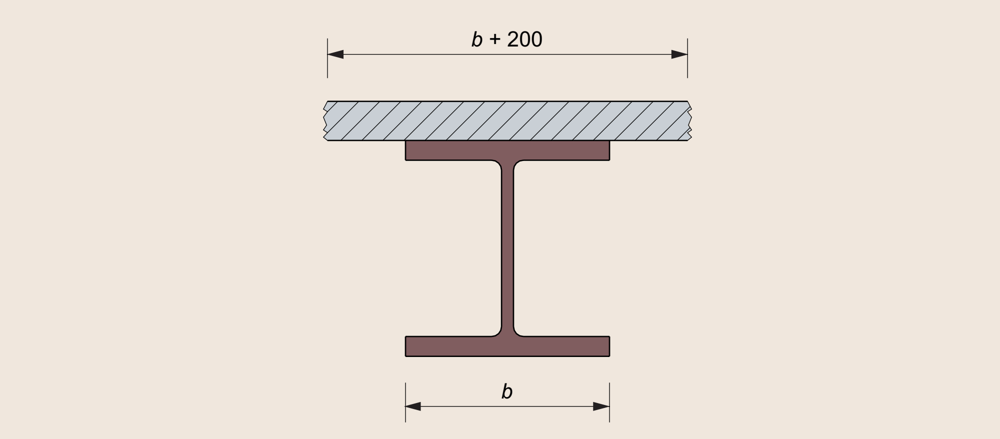

Figure 1
Column section with some partitioning attached

Section 7.8.13 In the design of key elements, the accidental loading should be applied in all directions, but only in one direction at a time. This means checking the key element column in bending about both the major and minor axes. The ordinary permanent and variable actions must also be taken into account and should be applied simultaneously with the accidental loading. However, the variable actions are reduced by applying factors ψ, as shown below.

Section 7.8.12 The key element design should also consider a reduced accidental action that corresponds to the blast resistance of the components (i.e. the partitioning) that are attached to the key elements.

The combination of actions for the design of key elements and attached components is expressed as:

BS EN 1990 Eq. 6.11b

$$\sum_{j\geq 1} G_{k,j} + A_d + \psi_{1,1}\, Q_{k,1} + \sum_{i>1} \psi_{2,i} Q_{k,i}$$

All of the calculations below relate to the column length between ground and first floor level (5 m storey height). In practice, all levels should be checked. The column section for the first storey is 300 × 300 mm. The total column self-weight is 50 kN for the full height of the building.

Actions

Accidental action – Maximum blast load:

A_d = Blast pressure × $(b + 200)/1000$ × column height

A_d = $34 \times (300 + 200)/1000 \times 5 = 85$ kN

Maximum moment, $M_{Ed} = 85 \times 5/8 = 53$ kNm

Accidental action – Reduced blast load:

For this example, it is assumed that the partitioning can resist a blast loading of 2.0 kN/m². At this loading, the whole of the partitioning remains attached to the key element. The total accidental load, applied to the key element is given by:

A_d = Blast pressure × column spacing × column height

$A_d = 2.0 \times 9 \times 5 = 90$ kN

Variable action:

In this example there is only one variable action to consider.

Table 3.1

For office floor loading, $\psi_1 = 0.5$

For roof loading, $\psi_1 = 0$

Therefore, there is no contribution from the roof variable action.

Design effects on column

Bending moment:

The total accidental action is 90 kN with the lower blast pressure compared to 85 kN when the full 34 kN/m² blast pressure is used. Therefore, the accidental action from the lower blast pressure should be used to design the key element, as this will be the critical design situation.

Maximum moment, $M_{Ed} = 90 \times 5/8 = 56$ kNm

Axial force due to permanent action:

$N_{G,Ed} = G_{k,j}$

= floor permanent action × floor area supported × number of storeys + column self weight

$G_{k,j} = 3.5 \times 9 \times 6 \times 10 + 50 = 1940$ kN

Axial force due to variable action:

$N_{Q,Ed} = \psi_{1,1}\, Q_{k,1} = \psi_{1,1}$ × floor load × floor area supported × number of storeys

$N_{Q,Ed} = \psi_{1,1}\, Q_{k,1} = 0.5 \times 6.0 \times 9 \times 6 \times 9 = 1460$ kN

Total axial force, $N_{Ed} = 1940 + 1460 = 3400$ kN

The key element column must be designed to resist the following combined effects:

Axial force, $N_{c,Ed} = 3400$ kN plus Major axis moment, $M_{y,Ed} = 56$ kNm and

Axial force, $N_{c,Ed} = 3400$ kN plus Minor axis moment, $M_{z,Ed} = 56$ kNm

Example 6 – Class 2b building – Transfer beam

Brief

What are the robustness requirements for the transfer beams in the floor arrangement shown in Figure 1?

The building is a 3 storey school building with transfer beams at the first floor level. The columns in the building are laid out on a 6 m × 7.5 m grid. The total weight of an internal column supported by a transfer beam is 8 kN. The dimensions of the transfer beam are: Depth = 1036 mm, Width = 310 mm, Web thickness = 30 mm and Flange thickness = 54 mm. All storeys are 4.0 m high, apart from the ground to first floor, which has a height of 5.0 m.

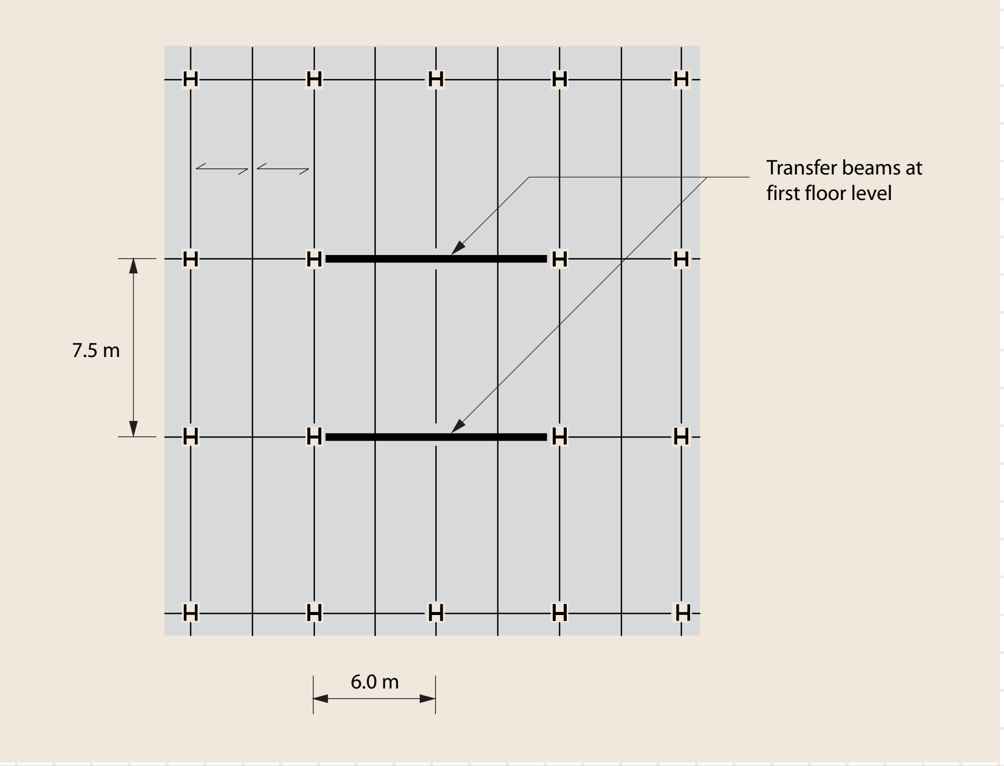

Figure 1
Beam arrangement

The floor loading is:

Permanent action, $g_k = 3.5$ kN/m^2

Variable action, $q_k = 6.0$ kN/m^2

The roof loading is:

Permanent action, $g_k = 3.5$ kN/m^2

Variable action, $q_k = 1.0$ kN/m^2

Building classification

BS EN 1991-1-7, Table A.1

School buildings greater than 1 storey but not exceeding 15 storeys are classified as Class 2b.

Robustness requirements

For Class 2b buildings there are three alternative methods that could be applied for robustness design; tying, notional removal, or key element. Only the tying and key element methods will be considered in this example.

Tying method

Robustness strategy

Section 9.4.2

For the tying method, horizontal ties and vertical ties should be provided. For transfer beams, the horizontal and vertical tying requirements should be modified from the general requirements presented in BS EN 1991-1-7.

Horizontal ties

Transfer beams, including their end connections, should be capable of sustaining a design tensile force T_{i} for the accidental limit state in the case of internal beams. The value of T_{i} is given by the following equation:

$T_{\mathrm{i}} = 0.8(g_{\mathrm{k}} + \psi q_{\mathrm{k}})sL + 0.5V_{\mathrm{c}}$ or 75 kN, whichever is the greater

where:

g_{k}	is the permanent action
q_{k}	is the variable action
s	is the spacing of ties
L	is the span of the tie
ψ	is the relevant factor ψ_1 or ψ_2. According to the NA to BS EN 1990. ψ_1 should be used
V_{c}	is the design loads from the columns supported by the transfer beam.

The column loads that are included in the calculation of T_{i} are based on the accidental loading condition, given below.

Section 7.8.13

$$\sum_{j\geq 1} G_{\mathrm{k},j} + A_{\mathrm{d}} + \psi_{1,1} Q_{\mathrm{k},1} + \sum_{i>1} \psi_{2,i} Q_{\mathrm{k},i}$$

In this case, there is no accidental action A_{d} and there is only one variable action to consider.

Table 3.1

For schools, floor loading (category C), $\psi_1 = 0.7$

For roof loading, $\psi_1 = 0$

Therefore, there is no contribution from the roof variable action.

Floor 2 loading into column:

$N_{\mathrm{col,floor}} = ((g_{\mathrm{k}} + \psi_1 q_{\mathrm{k}})sL)$

$N_{\mathrm{col,floor}} = ((3.5 + 0.7 \times 6.0) \times 6 \times 7.5)$

$N_{col,floor}$ = 346.5 kN

Roof loading into column:

$N_{col,roof} = ((g_k + \psi_1 q_k) s L)$

$N_{col,roof} = ((3.5 + 0.0 \times 1.0) \times 6 \times 7.5)$

$N_{col,roof} = 157.5$ kN

Total force in column:

$V_c = N_{col,floor} + N_{col,roof} + \text{self weight}$

$V_c = 346.5 + 157.5 + 8$

$V_c = 512.0$ kN

The required horizontal tie resistance T_i is:

$T_i = 0.8(g_k + \psi q_k) s L + 0.5 V_c$ or 75 kN, whichever is the greater

$T_i = 0.8(3.5 + 0.7 \times 6.0) \times 6 \times 7.5 + 0.5 \times 512$

$T_i = 533$ kN

The transfer beams, including their end connections, should be capable of sustaining a design tensile force of 533 kN.

Vertical ties

Section 9.4.2 For transfer beams, the vertical tying requirements should be modified so that columns supported by transfer beams are vertically tied to the transfer beam. The tensile resistance of the connection should be designed for the maximum tension that would occur in the accidental loading case if either one of the columns supporting the transfer beam were removed.

The required vertical tie resistance T_v is:

$T_v = (g_k + \psi_1 q_k) s L$

$T_v = (3.5 + 0.7 \times 6.0) \times 6 \times 7.5$

$T_v = 347$ kN

The connection between the transfer beam and the base of the column that it supports should be capable of sustaining a design tensile force of 347 kN.

Tying method

Additional structural provisions

Section 7.1.2

The recommended additional structural provisions relating to vertical bracing and anchorage of heavy units should be applied to this building. These do not directly relate to the transfer beam therefore they are not included in this Example. See Example 3 for the application of these additional structural provisions.

Key element design method

Robustness strategy

Section 9.4.3

BS EN 1991-1-7 does not require all transfer beams in Class 2b buildings to be designed as key elements. If the tying or the notional removal methods of robustness design do not demonstrate adequacy, the designer may chose to use the key element method.

In addition to designing the transfer beam as a key element, it is also recommended that the columns that support the transfer beams are designed as key elements. An example of column key element design is presented in Example 5. This Example will only consider the key element design of the transfer beam.

Section 7.2.2

Key elements should be capable of sustaining an accidental design action of A_d applied in horizontal and vertical directions (in one direction at a time) to the member and any attached components. The recommended value of A_d for building structures is 34 kN/m^2.

For the design of a transfer beam key element, the accidental action must be applied in 3 directions:

1. acting horizontally against the side of the transfer beam;
2. acting vertically upwards on the underside of the transfer beam and the floor slab that it supports;
3. acting vertically downwards on the top of the floor slab that the transfer beam supports.

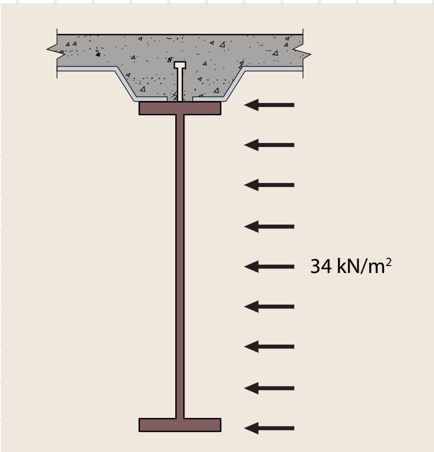

Figure 2 Horizontal accidental action on transfer beam

Horizontal accidental action

The horizontal accidental action on the transfer beam is shown in Figure 2. The first step is to determine whether the accidental action causes the beam to become detached from the slab.

Resistance of beam slab connection

The transverse resistance of the shear connection depends on the stud diameter height and configuration of the surrounding concrete. For this example, the resistance is taken as 50 kN. For move exact values, refer to EC4, noting the direction of the force. The studs are positioned at 300 mm spacing along the beam. Therefore, the connection resistance = $50 \times 1000/300 = 167$ kN per m length.

Lateral force on beam slab connection

The beam depth is 1036 mm. Therefore, the total lateral force is $34 \times 1036/1000 = 35.2$ kN per m length.

Therefore, the beam will remain attached to the slab.

The beam should be checked to determine whether the accidental action causes yielding of the web. The web thickness is 30 mm, the design strength of the steel is 345 N/mm^2.

The elastic bending resistance of the beam web (per m length) is given by:

$$M_{c,Rd} = f_y \times t^2/6$$

$$M_{c,Rd} = (345 \times 30^2 / 6) \times 10^{-3}$$

$$M_{c,Rd} = 51.8 \text{ kNm per m length}$$

The design moment applied to the web (per m length) is given by:

$$M_{c,Ed} = 34 \times h^2/2$$

$$M_{c,Ed} = (34 \times (1036/1000)^2/2)$$

$$M_{c,Ed} = 18.2 \text{ kNm per m length}$$

Therefore, the beam web can resist the effects of the accidental action.

Downwards Accidental Action

The downwards accidental action on the transfer beam is applied to the top of the slab supported by the transfer beam. It is assumed that the slab will remain attached to the transfer beam for this loading situation.

The transfer beam must be designed for all the loads applied in this accidental situation, as given by the combination below.

Section 7.8.13

$$\sum_{j\geq 1} G_{k,j} + A_d + \psi_{1,1}\, Q_{k,1} + \sum_{i>1} \psi_{2,i} Q_{k,i}$$

Loads from secondary beams:

$$V_1 = (g_k + \psi_1 q_k)\, s\, L$$

$V_1 \quad = (3.5 + 0.7 \times 6.0) \times 6 \times 7.5/2$

$V_1 \quad = 173 \text{ kN}$

Loads from column due to floor load:

$V_2 \quad = (g_k + \psi_1 q_k) s L$

$V_2 \quad = (3.5 + 0.7 \times 6.0) \times 6 \times 7.5$

$V_2 \quad = 347 \text{ kN}$

Loads from column due to roof load:

$V_3 \quad = (g_k + \psi_1 q_k) s L$

$V_3 \quad = (3.5 + 0 \times 6.0) \times 6 \times 7.5$

$V_3 \quad = 158 \text{ kN}$

Loads from column due to column self-weight:

$V_4 \quad = 8 \text{ kN}$

Total loads from column:

$V_{col} \quad = 347 + 158 + 8$

$V_{col} \quad = 513 \text{ kN}$

Section 7.8.13 To calculate the load on the transfer beam from the 34 kN/m^2 accidental load, the area that the accidental load is applied to may be reduced from the full area supported by the transfer beam. The reduced loaded area may be taken as $2.25 \times$ storey height squared. In this case $2.25 \times 4 = 9$ m squared i.e. 81 m^2. (The full area would be $2 \times 6 \text{ m} \times 7.5 \text{ m} = 90 \text{ m}^2$).

The area loaded by the accidental action is applied in the most severe position along the transfer beam. In this case, the load is applied at the centre of the transfer beam as shown in Figure 3.

Total loads from accidental action:

$A_d \quad = 34 \times 81$

$A_d \quad = 2754 \text{ kN}$

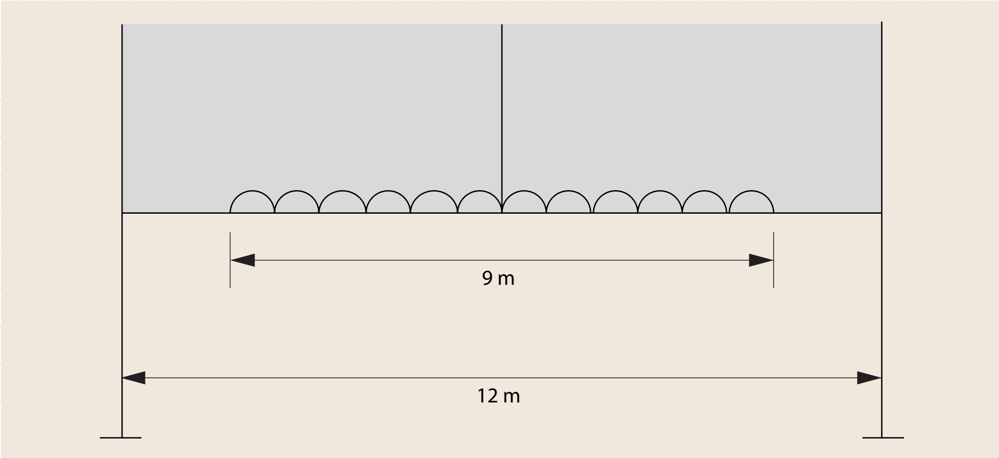

Figure 3 Accidental action applied to the transfer beam

Moment on transfer beam due to accidental action:

$M_{y,Ed,1}$ $= A_d$ (1.5 m + 0.25 × 9 m)/2

$M_{y,Ed,1}$ = 2754 (1.5 + 2.25)/2

$M_{y,Ed,1}$ = 5164 kNm

Moment on transfer beam due to secondary beams:

$M_{y,Ed,2}$ $= V_1 \times 12$ m /4 + $V_1 \times 3$ m

$M_{y,Ed,2}$ = 173 × 12 /4 + 173 × 3

$M_{y,Ed,2}$ = 1038 kNm

Moment on transfer beam due to column load:

$M_{y,Ed,3}$ $= V_{col} \times 12$ m /4

$M_{y,Ed,3}$ = 513 × 12/4

$M_{y,Ed,3}$ = 1539 kNm

Total design moment on transfer beam:

$M_{y,Ed}$ = 5164 + 1038 + 1539

$M_{y,Ed}$ = 7741 kNm

The transfer beam must be designed to resist a major axis moment of 7741 kNm.
The transfer beam is restrained by the floor slab and secondary beams.

Upwards accidental action

The upwards accidental action on the transfer beam is applied to the underside of the transfer beam and the slab supported by the transfer beam. This case can only be more critical than the downwards accidental action if the transfer beam becomes detached from the slab and becomes unrestrained. The first step is to determine whether the accidental action causes the beam to become detached from the slab.

Resistance of beam slab connection

The upward push-out value of the shear stud is assumed to be 10 kN. The studs are positioned at 300 mm spacing along the beam. Therefore, the connection resistance $= 10 \times 1000 / 300 = 33.3$ kN per m length.

Load on beam slab connection

The load on the beam to slab connection due to the accidental action is given by:

$F_1 \quad = 34 \times 7.5 = 255$ kN per m length

The permanent action of the floor slab acts in the opposite direction to accidental action and its value is given by:

$F_2 \quad = g_k \times 7.5$

$F_2 \quad = 3.5 \times 7.5$

$F_2 \quad = 26.3$ kN per m length

The resultant load on the beam slab connection is $255 - 26.3 = 229$ kN/m which is greater than the connection resistance of 33.3 kN/m. Therefore, the beam will become detached from the slab.

The load on the transfer beam will be the accidental action applied to beam width. The beam width is 310 mm. Therefore, the total load applied to the transfer beam is $34 \times 310 / 1000 = 10.5$ kN/m.

The resultant moment on the transfer beam, neglecting the loads from the column and slab is given by:

$M_{cy,Ed} \quad = 10.5 \times 12^2 / 8 = 190$ kNm

The transfer beam must be designed to resist a major axis moment of 190 kNm. The transfer beam is unrestrained.

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STRUCTURAL ROBUSTNESS OF STEEL FRAMED BUILDINGS

Structural robustness is an important consideration for the design of buildings so that the likelihood of disproportionate collapse, as a result of accidental actions, is limited. This publication provides design guidance for hot rolled steel framed buildings on the Eurocode strategies for structural robustness and designing for the avoidance of disproportionate collapse as required by the UK Building Regulations. Guidance on recommended good practice is offered where the Eurocodes do not include requirements or where they are not specific and are open to interpretation. The scope of this publication is limited to application in the UK and reference is made to the UK National Annexes as appropriate. In addition to the design guidance, six worked examples are included to demonstrate the application of robustness strategies to different classes of building.

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